WATER POWER

A technical journal devoted to the study of all aspects of Hydro-electric developments

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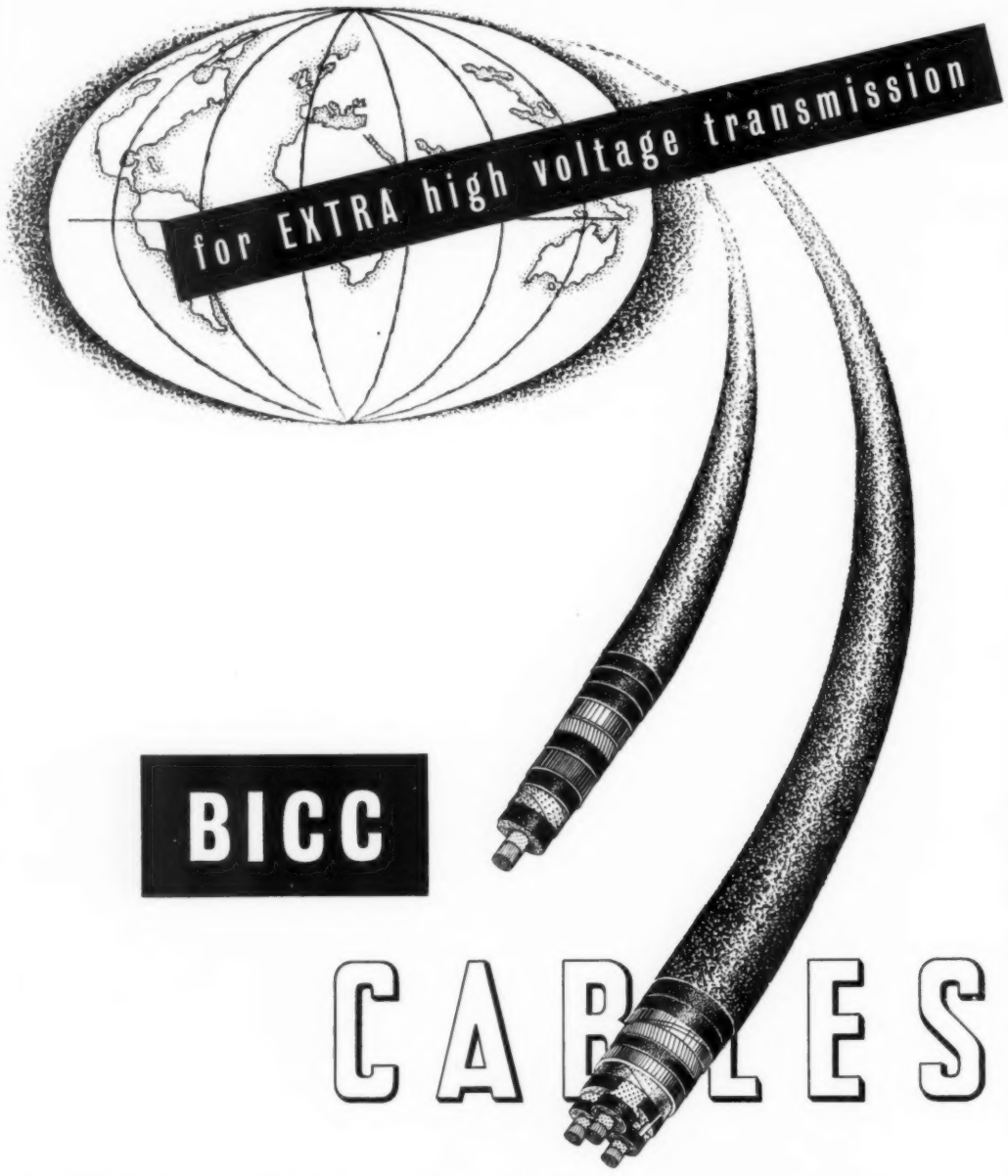

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WATER POWER

A technical journal devoted to the study of all aspects of Hydro-Electric Development

Electrical Preferences

ON the electrical side of hydro-electric development, the CIGRE Conference forms the unrivalled "Parliament" for free expression of views on all aspects of the generation, control, and transmission of power. The governing body of CIGRE has recently issued a statement about the reorganisation of the affairs of the Conference, which, of course, is held every two years in Paris. Not only are all the 17 groups of subjects divided into two categories, five of which are to be discussed at every Conference, and the remainder at every other Conference, but in addition the CIGRE Council has decided that certain subjects are "preferential" and that in view of the "great present interest" in these subjects, the National Committees should do all in their power to obtain papers which deal with them for the 1956 Conference.

In a moment we will indicate the nature of these preferential sections; we would only note in passing that there may be other views, perhaps at variance with those of the learned members of the CIGRE Council, as to which aspects of the electrical side of water-power exploitation are to be considered as of more immediate importance than others.

The "preferential" subjects for the 1956 Conference include, among the alternator group, the verification of temperature rise in windings, the dielectric testing of alternator windings, and the determination of temperature conditions within an alternator. On the transformer side, the CIGRE Council feels that there is still much to be said about chopped-wave testing, with particular reference to fault location and detection. This subject has been very widely discussed in recent years, and many observers thought that finality (if not satiation) had been reached at the 1954 session. However, it seems that the transformer specialists are still undecided on the solutions to this type of problem. There is also a request for papers on on-load tap-changing equipment and on transformer noise—a problem that is certainly becoming acute in those locations where more and more power has to be brought into urban areas, necessitating the siting of the step-down transformers fairly close to inhabited premises.

The preferential subjects among the circuit-breaker papers include the hoary problem of indirect testing. This should give rise to some interesting clashes of opinion between American and European specialists in this field, because in the United States great reliance is still placed on the bulk-oil single-unit circuit breaker, whereas practically all European designers produce multi-break or air-blast designs, to which indirect testing methods can be applied with much greater ease. Papers are called for on the determination of the natural frequency of supply networks, and here we take leave to wonder whether this is a choice which many engineers will regard as being of a preferential nature. We raise this query because the constant change in system characteristics and the incessant addition of further lines, together with increased interlinkage with neighbouring networks, makes us wonder if the painfully and laboriously gathered statistics relating to this network or that will be of the same permanent value as that which attaches to the majority of the papers emanating from each CIGRE Conference.

The Cable Section is asked to produce papers on the use of plastic insulation for high-voltage cables, and it will be interesting to see, next year, if this field shows sign of expanding. Aluminium for conductors and sheaths has now become almost a commonplace material and the papers requested on this subject will doubtless tend to deal more with experience of installations which by now will have been in service for several years, rather than with new developments. The ability of high-voltage cables to withstand internal surges is a matter on which relatively little is known, and cable engineers and operating staffs will doubtless welcome the papers which the CIGRE Council request on this subject.

The Cable Section is also asked to provide papers on "long-distance" submarine cables. The quotation marks are our own, because the word "long" is not clearly defined, and many engineers will think that

there is in fact only one really "long" submarine power cable in the world, that reaching out for 62½ miles across the Baltic Sea from the mainland of Sweden to supply the Island of Gotland. This cable, of course, operates on high-voltage direct current, and the installation has been extremely successful. The Conference recognise this by asking for papers on experience gained on this d.c. transmission and on the comparison between bulk transmission by a.c. and d.c. We wonder whether cable engineers, by 1956, will be able to bring news of further "long-distance" power cables operating on d.c., or whether there will perhaps be some suggestion as to a method whereby conventional three-phase a.c. power can be carried for a distance greater than the 20 or 30 miles which now forms the technical limit for this type of transmission.

It is perhaps a little surprising to find that the CIGRE Council regard papers on the calculation of towers to withstand broken conductors to be a subject in which there is much that is new and likely to be of interest to those attending the Conference. This aspect of transmission, together with the calculation of tower foundations (another preferential subject) is work in which we should have thought that little fresh material would evolve for some considerable number of years to come. There is, however, a request in the "Towers and Tower Foundations" group for papers on towers for radio-link transmission, and here there is no doubt that much work has been done both in America and in Europe, and there will be a wide audience for such papers if they materialise.

From the economic point of view there have been suggestions in recent years that the cost of overhead transmission links could be considerably reduced if the very high safety factors, based mostly on empirical calculations, could be considered from a more scientific standpoint, with the possibility of adjustments in a downward direction without any loss of reliability or safety. It is not therefore surprising to find a request for papers on this subject, although the aspects of it also mentioned—line insulation, wind pressure on conductors, and permissible operation temperature for conductors of various kinds—all appear to be integral parts of the main subject relating to the reduction in the cost of the line by the application of revised standards.

The formation and elimination of sleet and ice is a subject which has been well canvassed in past years and so is the use of bundle conductors versus single conductors; we should not have thought that these were regarded as preferential matters for discussion. They are, however, included in the request for papers in the extra-high-voltage transmission section.

Relaying and stability is a subject on which there is always something new to be said, and the complication complex which for several decades has attacked relaying engineers in most countries now appears to be challenged by a request for papers on standardisation of relays; and it is hoped that the great need for simplification in this field will receive proper emphasis. The protection of lines equipped with series capacitors is very rightly put forward as a priority question, since the important increase in transmission capacity brought about by the use of these devices has been one of the most significant features of electrical transmission during recent years.

Here, then, is a bird's-eye view of what is considered as being most important by the CIGRE Council, representing all electrically developed nations of the world; and there is no doubt that such a grading operation will have considerable value not only at the Conference itself but also by directing research efforts into directions that responsible engineers consider to be most desirable at the present time.

Work Begun on Kariba

BY making an immediate start on certain sections of the preliminary work on this great project it is hoped to begin power production a year sooner than was originally considered possible. This work includes a footbridge of the suspension type across the Zambesi to facilitate bank-to-bank communication, the driving of a diversion tunnel, and the construction of a cofferdam, all of which are estimated to be finished during the next low-water season.

The report of Messrs. Andre Coyne and Jacques Tisne, the French consultants nominated by Electricité de France at the request of the Kenya Government, has now been published. This report expresses a very favourable opinion of the site chosen for the dam, immediately upstream of the gorge, and recommends that it be accepted as the best available. The dam will create a lake 230 miles in length and having a surface area of 1,600 sq. miles; it will be 400 ft. high and have a crest length of about 2,000 ft. Tenders for the preliminary work have already been invited and it is understood that tenders are now being prepared for the civil-engineering work and for the turbines and electrical equipment.

Ontario Hydro Progress

IN its Annual Report for 1954, the Hydro-Electric Power Commission of Ontario reveals that the dependable peak capacity of its system was raised by no less than 16 per cent. over that of 1953, to a figure of 4,135 MW, providing some respite from the almost continuous pressure of increasing power demands since the end of the war. Favourable water conditions also contributed to the success of the year's operations. The total energy production for commercial load purposes during the year amounted to 22,386,456,876 kWh, of which 18,077,666,964 kWh was produced in the 65 hydro and six thermal stations owned by the Commission. The net output of all resources increased by 7 per cent. over the 1953 figure, and there was a 5·6 per cent. increase in the number of ultimate consumers served.

The year 1954 was a historic one for Ontario Hydro in that the Sir Adam Beck-Niagara Generating Station No. 2 was officially opened on August 30 by Her Royal Highness, The Duchess of Kent. Earlier in the same month, international ceremonies had signalled the official start of construction on the St. Lawrence power project. Two additional units were placed in service at the Pine Portage plant on the Nipigon River, bringing its capacity to 118·3 MW, and marked progress was made in the building of a new 42·1 MW development at Manitou Falls on the English River. The latter two sources of power will assist materially in meeting the ever-growing demands for power in the North-western part of the Province. Significant progress was also made on the construction of remedial works in the Niagara River above the Falls

Official ground-breaking ceremonies for this undertaking were held on June 2.

Looking ahead to the completion of the St. Lawrence development, the Commission directed its research activities in large measure to the St. Lawrence power project. For future system protection, studies regarding nuclear power were carried forward in association with Atomic Energy of Canada Limited.

The Report records with the utmost regret, which will be widely shared, the death in an air disaster last January of Mr. Robert H. Saunders, C.B.E., Q.C., the Commission's Chairman from 1948 to 1955.

Transformers for St. Lawrence

THE English Electric Co. Ltd. has just received the order from the Power Authority of the State of New York for two large transformers to be used in connection with the St. Lawrence Power Project now being erected on the St. Lawrence River jointly by the Canadian and United States Authorities. For the Canadian side the English Electric Company of Canada Limited is building in Toronto the water turbines to the designs of The English Electric Co. Ltd. in England.

On the American side, part of the power generated will be stepped-up to 115 kV and part to 230 kV. The transformers which the Company is now to build will couple these two busbars together for the interchange of power. They have an output of 120,000 kVA with a voltage ratio of 230/115 kV with the windings auto-connected. A tertiary winding is provided for the supply of power at 13·8 kV. They are three-phase units and the cooling is of the OFW type (pumped oil circulation through water coolers). The main windings will be impulse tested at a level of 900 kV. These transformers are similar to an 80,000 kVA unit built by the Company and now in service at the Folsom dam in California. They will be arranged so that a group of four generators is connected to each bank of transformers.

Other dollar orders secured from the U.S.A. for transformers comprise 125,000 kVA three-phase units for the City of Seattle; 310,000 kVA three-phase banks for the Chief Joseph dam on the Columbia River and a 250,000 kVA three-phase bank for the Bonneville Power Administration. The units for Seattle have been delivered and the first units for the Chief Joseph dam are on site, ready for service. The installed capacity of the Chief Joseph dam station is second only to that of the Grand Coulee dam station, the largest in the U.S.A. The power house on the Chief Joseph dam is the longest in the world, and is designed to accommodate twenty-four 67,368 kVA (nominal) hydro-electric generators. Incidentally, both these stations are on the Columbia River.

All the transformers mentioned in this paragraph are for 230 kV service.

Tasmanian Hydro-Electric Station

THE Tungatinah hydro-electric station, Tasmania, was formally opened recently by Sir William Slim, Governor General of Australia. This station was described in our December, 1954 issue and marks a further step in the development of the water-power resources of Tasmania, bringing the installed capacity of hydro generating plant up to approximately 400,000 h.p. Further developments are expected to raise this figure to nearly 800,000 h.p. by 1960.

The interior of Tasmania is an extensive plateau with an area of approximately 1,500 sq. miles, at an altitude between 2,000 and 4,000 ft., and a relatively high rainfall. Average annual rainfall for the western perimeter is about 80 in. and the area contains several lakes which provide means for the storage of water at relatively low cost.

The Tungatinah scheme is the largest of several completed to date. Most of the plant installed was described in WATER POWER of December last but no mention was made of the transformer installation. This comprises fifteen single-phase units each of 10 MVA, manufactured by the Brush Electrical Engineering Co. Ltd. The units are arranged in five 3-phase banks and step up the generating voltage of 11 kV to 110 kV for transmission. The arrangement of single-phase units was adopted because of the difficulty of transporting 3-phase units to a site 2,000 ft. above sea level and to the bottom of a relatively narrow gorge with very steep approaches. The low-voltage sides of the transformers have delta connections with star connections on the high-voltage sides. The sides are of the standard Brush core-type design, oil immersed, and naturally self-cooled.

The abundant and cheap supply of electricity has attracted to Tasmania a number of important industries for which power costs constitute a major part of the total production costs. The chief is the Electrolytic Zinc Company of Australia, whose undertaking near Hobart absorbs 67,000 h.p., shortly to be increased to 97,500 h.p. The recently-established Australian Aluminium Producing Unit, near Launceston, will require about 35,000 h.p. and there are several other industrial undertakings taking a further 20,000 h.p. These industrial developments in association with the highly developed fruit-growing industry give a good balance to the economy of the island. The electrification of the rural areas of Tasmania is also highly developed, and electricity is now available to 97 per cent. of the island's population.

Brazilian Traction Report

THE Annual Report for 1954 of Brazilian Traction, Light and Power Co. Ltd., once again reveals encouraging progress despite a serious drought. No less than 420 MW of new generating capacity, including the 200 MW steam plant at Piratininga, was brought into commission during the year, and although the reservoirs were drawn down to dangerous levels, this new capacity not only averted a serious power crisis but enabled the company to supply 12 per cent. more power to consumers than in 1953.

In the Rio system river flows and run-offs during 1954 were even smaller than those of 1953 and were the lowest recorded in over 30 years. The Lajes reservoir at its lowest point, at the end of September, was down to 15·9 per cent. of its useful capacity. The flow of water in the Paraíba River during the year was only 53 per cent. of the average for the last 34 years. The Rio system, however, generated 3,221,590,200 kWh in 1954, an increase of 42 per cent. over 1953. This large increase was made possible by the addition during the year of four generating units of 65,000 kW each to the Nilo Peçanha underground power plant, bringing the rated capacity of this plant

to 330,000 kW. The fourth set of pumps at the Vigário and Santa Cecilia pumping stations in the Paraíba-Piraí Diversion project were put into service to provide the additional water needed to utilise the full capacity of this plant.

The larger output from the Rio system and the ability to transmit energy at 60 cycles directly from the Nilo Peçanha plant to the São Paulo system permitted the transmission of approximately 440 million kWh to São Paulo during the year.

Rainfall in the São Paulo area was again very much below average in 1954, causing a serious shortage of water. River flows were 70 per cent. of average and the water level in the Billings reservoir, the largest in the system, dropped to the lowest mark ever recorded; at the end of September 1954, the reservoir contained only 13 per cent. of its useful capacity. The emptying of the reservoir was only averted by the bringing into service of the Piratininga steam-electric plant and by the supply of electric energy from the Rio system.

Work was completed during the year on the raising of the height of the Edgard de Souza dam, and the construction of a new dam at Pirapora on the Tietê River was started. The Pirapora dam will impound additional water from the Tietê and Juquerí Rivers and, with the installation of a pumping station at the Edgard de Souza dam, will provide additional water for the Billings reservoir. Substantial progress was made in the construction of the new underground hydro-electric plant at Cubatão. It is now hoped that the first of four 65,000 kW units will be brought into service during the first quarter of 1956 and three more units later in that year.

Ybbs-Persenbeug Generators

THE contract for the construction of the first of the 45,000 kVA generators for the Ybbs-Persenbeug power station of Osterreichische Donaukraftwerke A.G. (Austrian Danube Power Co. Ltd.) was awarded to ELIN last year and this award has now been extended to cover all six generators. These machines will be the largest of any run-of-river plants in Austria, each weighing 405 tons and requiring a housing 14 m. in diameter. The first will be put into service in 1957 and it is expected that the whole plant will be completed by the middle of 1959. An interesting point is that the cooling water for these machines will be circulated around a nearby housing estate during winter months on an ingenious system devised by the manufacturing firm, a practice which might profitably be extended where circumstances permit. We hope to give full particulars of this plant in due course.

Contracts Placed for Orrin Scheme, Scotland

THE North of Scotland Hydro-Electric Board have now placed the first of the main contracts for their Orrin hydro-electric scheme (Constructional Scheme No. 29). Duncan Logan (Contractors) Limited, Muir of Ord, Ross-shire, have been awarded the contract to build the dam, which will be sited five miles upstream from the Orrin Falls. This dam will be 150 ft. high and 1,000 ft. long. Whatlings Limited, of Glasgow, have been given the contract to build the 16,000 ft. long tunnel which will carry water from the new reservoir formed by Orrin dam to a power station in Strathconon, near Achonokie, on the southern shore of the small loch created by Torr Achilty dam. This power station will have a total capacity of 18,000 kilowatts. The value of these contracts is approximately £4,300,000.

The Orrin scheme, which is the third stage of the Conon Valley development, will harness the water-power resources of an area extending to 54 sq. miles and draining into the River Orrin. The annual production of electricity from the scheme is estimated to be about 80 million kWh.

British Nuclear Energy Conference

IN the light of recent rapid developments in the technology of nuclear energy, and the increasing demand for a common ground between scientists and engineers where these developments can be discussed, an organisation has been formed by the Institutions of Civil, Mechanical, Electrical, and Chemical Engineers, and the Institute of Physics, to satisfy this need. This will be known as the British Nuclear Energy Conference, whose affairs will be managed by a Board consisting of three representatives from each of these societies. The Chairman is Sir Christopher Hinton, and Sir John Cockcroft is one of its distinguished members. The Secretary is Mr. Alexander McDonald (Secretary of the Institution of Civil Engineers, Great George Street, London, S.W.1).

The five societies will arrange for the presentation of Papers dealing with nuclear energy subjects, and all members of the societies will be able to attend and take part in the discussions.

The Conference will publish a Journal about four times a year containing records of the Papers, discussions, symposia, and conferences conducted by the Board. The Board proposes to hold its inaugural meeting in the autumn at which a symposium of lectures will be delivered on the technology of nuclear energy, and it is also proposed to promote national and international conferences from time to time.

Battelle Memorial Institute

ROBERT Q. WILSON has been appointed the United Kingdom representative for Battelle Memorial Institute, well-known international industrial research organisation. He will be in charge of the Institute's recently registered office in London, Battelle Institute Ltd., 46, Bryanston Street, W.1., and will maintain liaison for British firms sponsoring research at Battelle laboratories in Columbus, Ohio; Geneva, Switzerland; and Frankfurt/Main, Germany.

Formerly assistant chief of the Columbus laboratory's division of chemical engineering, Mr. Wilson has been a member of the Battelle research staff since 1948. The London Office was established for the convenience of firms in the United Kingdom in need of contract research facilities of the type provided by the Institute's laboratories. In addition to Battelle Institute Ltd., other offices are maintained in Paris, Milan, and Madrid. Battelle, founded in Columbus in 1929, is the world's largest not-for-profit industrial research organisation. It conducts research for industry on a contract basis in all fields of industrial science. In 1954, it conducted more than $15 million worth of research for industrial and governmental sponsors throughout the free world.

Some Geological Aspects of Catchment Areas and Reservoir Sites

By DR. J. E. RICHEY, M.C., F.R.S., F.R.S.E.,
Consulting Geologist, Edinburgh

PART ONE

IT is well known that problems in hydraulics are often interrelated with geology, and this is especially so in the case of hydro-electric schemes. In general, the engineer's plans are closely bound up with the form of the ground, which is itself an expression of the local geology. In particular, the composition and structural disposition of rocks and superficial deposits are subjects, amongst many others, with which the engineer has constantly to deal. In regard to schemes involving the impounding of water in reservoirs, he has to consider in the first place the effectiveness of a catchment area for providing the water he requires for storage. In his investigations he has to take into consideration a number of factors, chiefly the amount and incidence of the rainfall and its variations at different periods of the year. Among other questions to which definite answers are required are the amount of the run-off carried by streams and rivers, the variations of the supply during dry periods; and in arid regions or those with seasonal rainfall, the rate of evaporation governing losses from the surface of reservoirs. The reservoirs themselves often present a variety of problems, geological as well as hydraulic, but as a rule the investigation of reservoir sites and catchment areas is the least exacting part of the geological work required for hydraulic schemes involving dams and tunnels.

Catchment Areas

The factors affecting the suitability of a catchment area are bound up chiefly with climate, topography, vegetation and geological conditions. The nature of the climate and the topography determine, conjointly, the amount and incidence of the rainfall, as an example of which an isohyetal map of Tasmania is reproduced (Fig. 1). Climatic conditions combined with types of vegetation control the proportion of the rainfall which is returned to the atmosphere by evaporation directly from the ground and from foliage, and by transpiration of plants and trees. Topographical and geological factors regulate the amount of the direct run-off and the proportion available for dry-weather flow of streams and rivers. Geological conditions decide the proportion of rainfall which will pass underground into the rocks and may be permanently lost to the catchment area.

Permanent losses due to the impounding of water below ground in permeable rocks and its escape elsewhere are seldom of importance unless in regions of low rainfall and where the water table is deep. Exceptions due to geological conditions are tracts of country composed of limestone where much of the rainfall passes downwards to make its way along underground rivers and channels. High-level areas of sandstone such as the Yorkshire moors are also subject to surface dryness, and much rain water may percolate into sandstones where the surface soils are permeable.

Geological conditions affect the dry-weather flow of streams and rivers and are worth considering before these waterways are gauged over a period of time. Bare rock, or rock with thin "soil," or superficial deposits of clay, especially when combined with steep gradients, leads to quick run-off and incidental periods of swollen rivers and flooded low ground alternating with greatly reduced supplies during dry spells. In contrast, extensive tracts of such permeable "drift" material as sand and gravel, many residual soils, and highly porous sandstones, may constitute valuable reservoirs of ground water, a proportion of which is often returned to a drainage system during rainless periods. Ground water moves slowly through permeable materials, and this is a guarantee of persistence of dry-weather flow.

Within the same catchment area contrasting conditions may occur, one river being "flashy" and the valley prone to flooding, whereas another may have a very equable flow. Examples of the two types are the River Alwen in North Wales and the Avon in the East Cairngorms, north-west of Braemar, Scotland. Both catchment areas are of about the same size. Yet the Alwen has a dry-weather flow of only 0·055 cusecs. per 1,000 acres, and the Avon 36 times this amount, that is 2·0 cusecs. per 1,000 acres.*[1] Both catchment areas are watertight, and the difference between them is directly related to the superficial geological conditions. The Avon basin is lined with much fissured and decomposed granite, which is permeable and so holds water during droughts. The Alwen valley is covered with boulder clay overlying Silurian shales and grits (greywackes) and its surface is practically impermeable.

The return of water to the surface from deep-seated storage in rocks is a factor to be considered in certain cases, wherever the geological conditions are favourable for the occurrence of perennially flowing springs. Underground aquifers, as a matter of normal procedure, are tapped by boring for irrigation purposes and human consumption, other than for the creation of hydro-electricity. A case from Algeria where the French hydrologists, geologists and engineers are engaged in utilising a sub-surface supply for the production of electric current is of special interest, in view

* The small superior numbers refer to a list of references given at the end of this article.

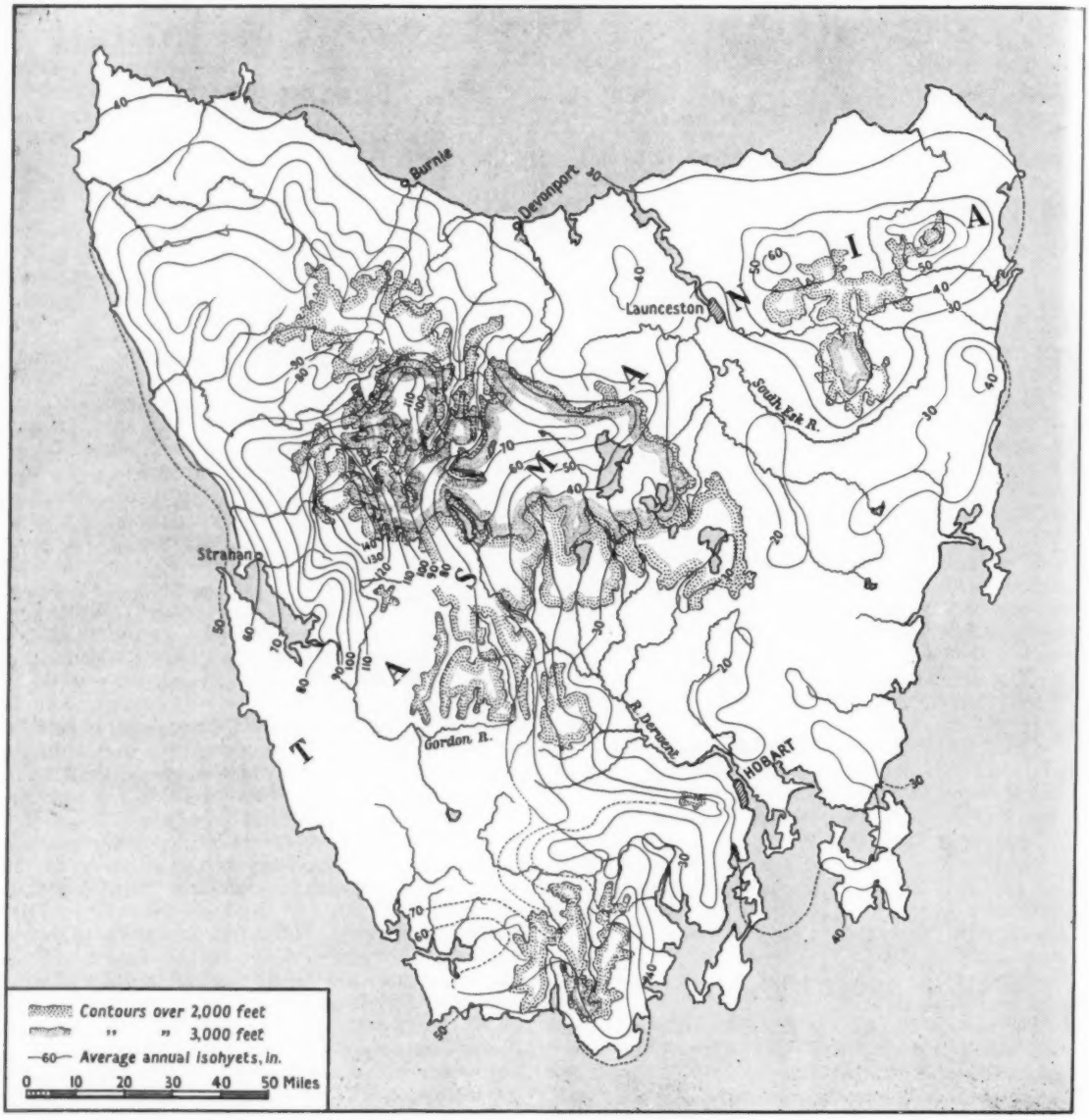


Fig. 1. Isohyetal map of Tasmania

of the unusual source upon which it is proposed to draw. This is the Chott Chergui project, on the high tableland of Western Algeria[2] between the Tell Atlas and the less bold Saharan Atlas to the south. The Chott is a vast closed basin in arid country, a salt-marsh the size of an English county, with a catchment area as large as Switzerland. The amount of the evaporation is enormous and the source of supply is largely artesian water which is drawn up to the surface, the aquifer being thick Cretaceous and Jurassic limestones which underlie much of the catchment area. The proposals are to tap the fresh artesian water underlying the superficial salt-laden ground by means of bores, and to extract in this way from the underlying aquifer no more than the amount of water which would otherwise be drawn to the surface and lost by evaporation. The artesian water is then to be conveyed, by gravity and by pumping, along surface aqueducts, tunnels and pipe-lines to the coastal plain for both irrigation and the production of hydro-electricity. The total available head of water is over 2,300 ft. (Fig. 2) and this will be handled in three stages. The main points bearing upon the feasibility of the scheme were the subject of extensive tests and include the following: the mean annual rainfall (11 in.); whether the basin was watertight; the infiltration coefficient within the area; the direction and speed of water movement in the Chott; and the annual evaporation losses. From the results of these investigations it was estimated that with bore-holes 150 to 300 ft. below the surface of the Chott it may eventually be possible to obtain a flow of 700 cusecs. or even

1,000 cusecs. This is equal to the amount of water flowing down the Thames at London during summer. With a flow of 460 cusecs. 400 million kWh of hydro-electricity could be generated yearly—a quantity which would almost double the present total electricity production in Algeria. It should be mentioned that some of the details given above from the publication quoted, may, more recently, have been modified by the French engineers.†

† The author is indebted to Professor Henri Termier, University of Algeria, for permission to reproduce the schematic map and sections shown in Fig. 2.

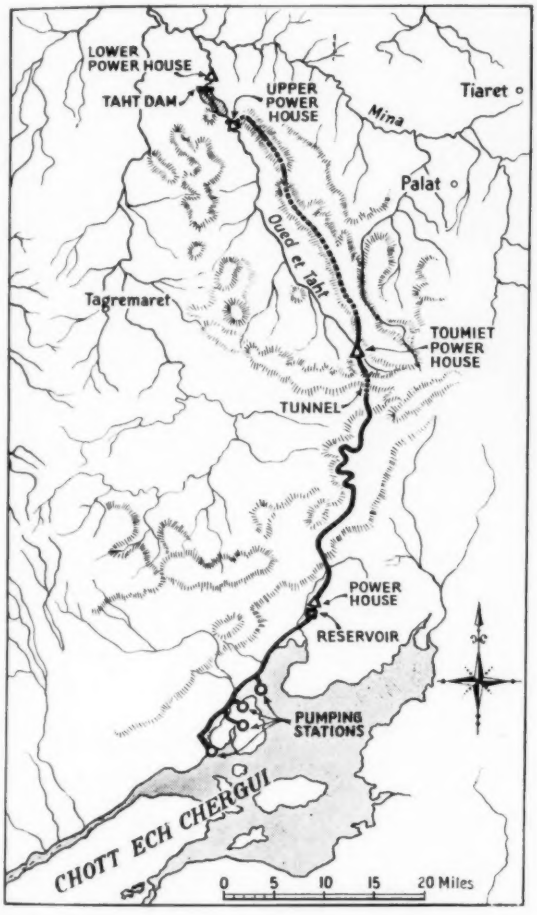


Fig. 2. Chott Chergui, schematic map and section

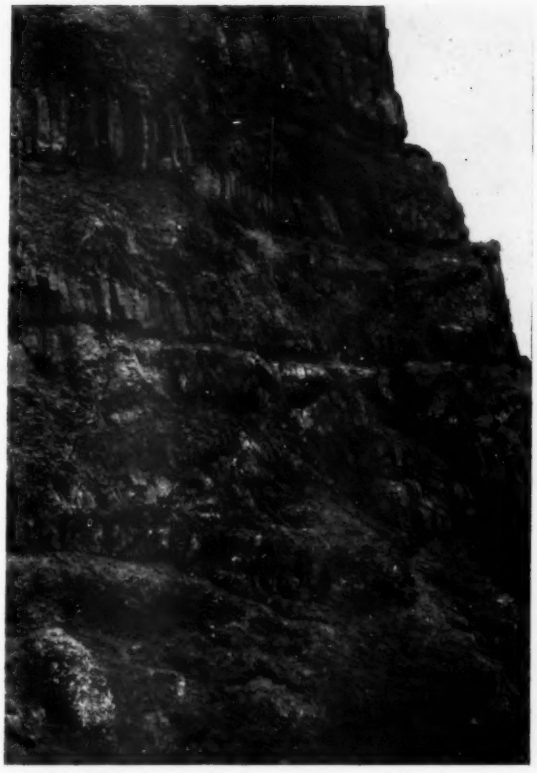
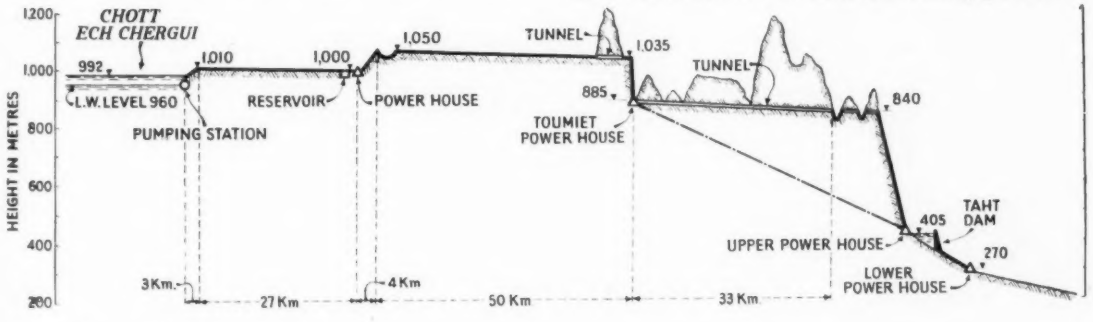


Fig. 3. Well-developed joints perpendicular to the bases of lava-flows constitute means of leakage from reservoirs as well as sources for underground supplies. Tertiary basalt lavas on coastal cliff, near Carsaig, Island of Mull, Scotland. The junctions between successive layers can be clearly distinguished. (Geological Survey Photo C. 2733)

Reservoir Sites

The main requirement is that the floor and sides of a reservoir should be watertight at all points. So the materials lining the reservoir must either be impermeable or structurally so arranged with reference to impermeable rocks or "soils" that any undue leakage is prevented. Permeable sand and gravel overlying clay or impermeable rock are safe provided the clay or rock extends up to the level of the water-table in the permeable deposits imposed by the top-water level of the reservoir. A permeable lining of this kind is an asset as it provides extra storage space for water which will be returned to the reservoir during periods of



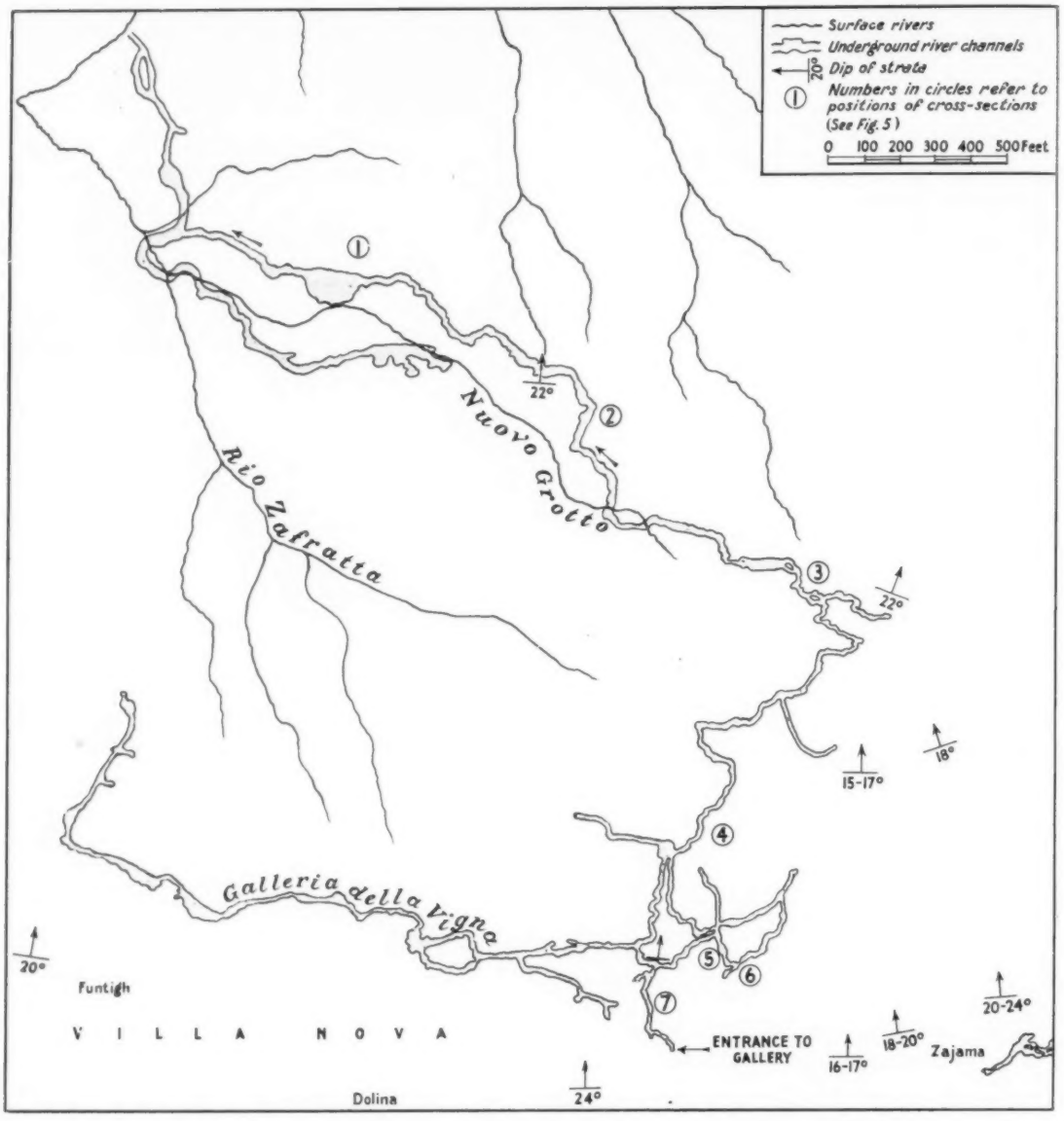


Fig. 4. An ancient underground river channel (see cross sections in Fig. 5)

drought or whenever the top-water level of the reservoir is lowered. It is a source of dry-weather supplies.

Landslips

The raising of the water-table that follows the filling of a reservoir may cause instability of valley-sides, especially where unconsolidated deposits rest upon steeply inclined rock surfaces. Where permeable superficial deposits such as sand and gravel or sandy moraine overlie the rock this effect may be most marked. With impermeable materials such as clay, the balance of stability may be upset and sliding take place along steep contacts with rock. A normal cause of landslips along rivers is the undercutting of a steep bank of superficial deposits by erosion at the convex side of river-bends. Extensive undercutting is less usual in the case of lakes. Another possible reason is illustrated by a case in Scotland. Landslips of moraine involving a road occurred on the steep western side of Loch Ericht, in Inverness-shire. The level of the loch had been altered by the construction of dams at both its ends, but this raised the water level only by 25 ft. and the draw-down of loch-level was seasonal; some other cause therefore seemed to be responsible, and deforestation of the slope appeared to be the most likely as the removal of the trees allowed ground water to percolate more freely into the moraine and down to rock-head.

Silting

The transport of sediment by streams and rivers is a geological process, strictly speaking, but it is of

special concern to the hydraulic engineer. As with lakes, the silting-up of reservoirs is a constant process and will sooner or later seal their final fate. The silting problem in reservoirs is a matter for the engineer to deal with, but in all too many cases the problem has proved insoluble. In nine reservoirs in the United States a survey established the disconcerting fact that the average amount of silt deposited approximates to 2·5 per cent. of the total reservoir capacity, per annum, the percentage for individual cases varying from 0·8 to 7·09.[3] Even for such a vast reservoir as that created by the Boulder Dam across the Colorado River a useful lifespan of only 200 years has been estimated. An instructive paper by Dr. D. J. Burdon may also be referred to, dealing with reservoirs in Cyprus.[4] His results show the relationship for eight reservoirs in this area between the percentage loss of capacity per year and the ratio of storage capacity to catchment area. He also describes briefly some methods of controlling the silting of reservoirs. In view of the importance of the silt problem, the transport of rock material by rivers has been the subject of much study by both engineers and geologists and some further papers dealing with it are appended for refer-

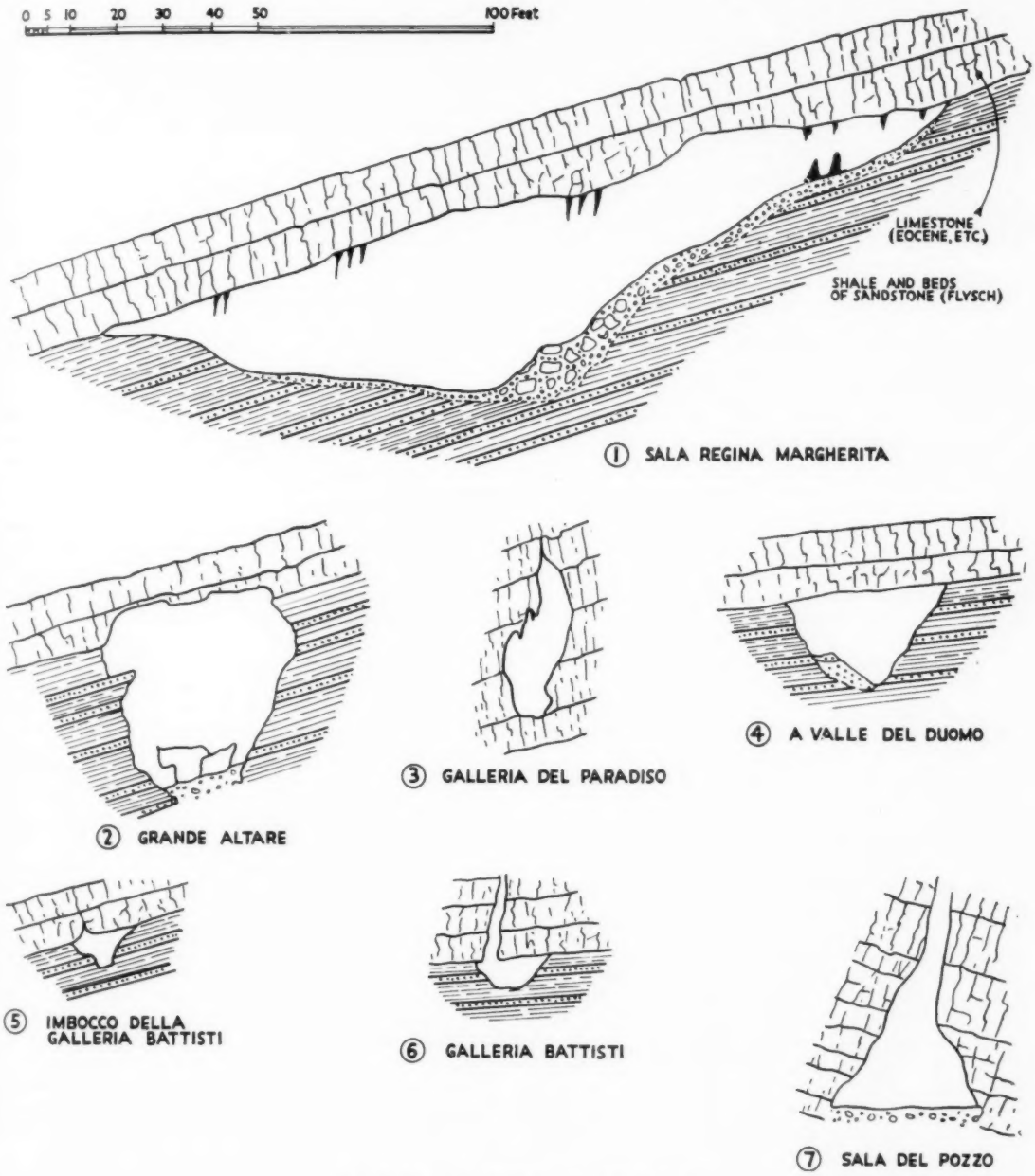

0 5 10 20 30 40 50 100 Feet
LIMESTONE (EOCENE, ETC.)
SHALE AND BEDS OF SANDSTONE (FLYSCH)
① SALA REGINA MARGHERITA
② GRANDE ALTARE
③ GALLERIA DEL PARADISO
④ A VALLE DEL DUOMO
⑤ IMBOCCO DELLA GALLERIA BATTISTI
⑥ GALLERIA BATTISTI
⑦ SALA DEL POZZO

Fig. 5. Cross section relating to Fig. 4

Fig. 6. The Smoo River disappears down a "swallow hole" in Ordovician limestone near Durness, N.W. Highlands of Scotland. (Geological Survey Photo C. 3126)

ence.[5] Two definitions may be given here. The *capacity* of a stream is the maximum load which can be carried in suspension. Its *competence*, which includes the *tractional* load moved along the stream-bed as well as the *suspended* load, is the ability of a stream to transport load in terms of dimensions of rock-particles. Rubey[6] gives the rule: "the weight of the largest debris moved by a stream varies as the sixth power of the 'bed' velocity." In regard to capacity, the same author states that "in a stream free to pick up much sand and gravel as its velocity is increased, the unit-width load will vary roughly as the third power of the 'bed' velocity of the stream."

A special feature regarding silting is the aggradation (deposition of sediment) which occurs upstream of a reservoir. A delta built out into a reservoir by an incoming stream will lead to aggradation upstream, and the effect will spread very rapidly upstream.[7] In contrast, where existing lakes or inland lochs are utilised as reservoirs, draw-down will reverse the situation upstream, and erosion of existing alluvium will take place. A recent example from the Scottish Highlands comes from Loch Fannich in Ross-shire. There, it is expected that erosion will eventually be adjusted by the deposition of new sediment brought down by incoming streams.

Leakage from Reservoirs

Of special importance is the question of leakage from a reservoir. Cases have arisen where the existance of permeable material below top-water level, both rock and "soil," has not been suspected until after a reservoir has been filled. Detection of any possible points of leakage may be the duty of the geologist, but their assessment is usually the responsibility of the engineer, who has to devise remedial measures to meet the situation. In areas of metamorphic and igneous rocks dangers of leakage are much more remote than in the case of sedimentary strata. A majority of metamorphic rocks are impervious, and the chances of leakage taking place along joints are usually negligible. Areas of igneous rocks such as granite and gabbro are equally safe.

Leakage Through Lava Flows

Lava flows are open to suspicion for various reasons. Vesicular upper portions of flows in cases where the little cavities are unfilled with minerals to form amygdales are notable water-carriers. Especial attention therefore has to be given to recently erupted lavas in which the vesicles are unfilled. In many areas of modern volcanoes such as Hawaii, lavas are sources of copious underground supplies of water. Lower solid portions of flows, too, carry water on account of their abundant joints, caused by shrinkage during cooling, which are usually arranged at right angles to the bases of flows (Fig. 3).* In ancient volcanic terrains, it is through these joints that the water percolates, often to issue in the form of springs at the base of the flow, the soft contemporaneously weathered upper portion of an underlying flow forming an impervious stratum to prevent any deeper percolation. Basalt lavas in particular are prone to carry water for one or other of the above-mentioned reasons, and there is also the possibility of leakage through open lava tunnels in certain terrains. Silicic, or acid, lavas are less commonly good aquifers; though often porous, they are not traversed by joints as extensively as basalts. A remarkable exception is a sheet of porous rhyolite and the glassy variety, obsidian, on the side of the Snake River Canyon, Idaho. At the junction of the lava sheets and underlying impervious rocks large quantities of water issue at several points. It has been estimated that these "Thousand Springs" yield 190 cusecs., which is equal to the amount required for the supply of New York City. Recent lavas of late Tertiary or subsequent date, intercalations of permeable sediments between lava flows or underlying a series of flows, may especially lead to dangerous leakages. In contrast, beds of volcanic ash are usually impermeable, although there are notable exceptions.

For one or other of the reasons stated above, where a "perched" water-table occurs in volcanic country, there is a liability of leakage around the edges of an impervious layer underlying a reservoir, or a dam. In areas with low rainfall, the main water-table may be situated at some depth below a water-logged valley floor, and when the water level is raised artificially by a dam the escape of water around the resulting

* Figs. 3, 6 and 7 are from Crown Copyright photographs and permission to use them in this article has been kindly granted by The Controller, H.M. Stationery Office.

Fig. 7. Joints in limestone widened by solution extend mainly down slope, so that underground flow of water is transverse to the valley. Surface of Ordovician limestone near Broadford, Isle of Skye, Scotland. (Geological Survey Photo C. 3116)

"perched" table into an underlying dry zone is always a possibility which must not be overlooked.

Leakage through Sedimentary Rocks

Sedimentary rocks fall into three main groups, namely, arenaceous such as sandstone, calcareous such as limestone, and argillaceous such as shales and mudstones, the final term being applied to consolidated muds without the closely spaced bedding planes characteristic of shales. The first two types are often permeable, and any occurrences below top-water level of a proposed reservoir call for investigation. Close-textured dark sandstones known as greywackes, and those consolidated with an argillaceous cement, are impermeable. But the more usual white, yellow and red quartzose kinds, with little cementing material or freely traversed by joints, are water-carriers. Joints in most rocks are open fissures at and near outcrops and become tight at relatively slight depths. In the case of sandstones, however, open fissures at depth are probably prevalent, and underground flow of water appears mainly to take place along them. An instance is on record from Derbyshire, England, of open joints up to 9 in. in width in sandstone occurring 500 ft. in from a valley-side and at a depth of 250 ft. vertically below the surface.[8] These fissures, however, appear to have resulted from gravity-settling of the valley-sides. Conglomerates on the other hand are usually poorly jointed rocks and also their pebbles are often bound together with a well-cemented matrix; this at least has been the writer's experience, apart from poorly consolidated pebble beds of Tertiary age.

Limestones are the most likely rocks to cause leakage from reservoirs. Although in themselves often close grained and impermeable, their jointing is apt to become enlarged through solution by percolating water, and the formation of characteristic underground channels is often a common feature of limestone country. Underground rivers may also erode channels in non-calcareous strata underlying limestone, the water being supplied from the limestone aquifer. A recently investigated example may be quoted, from Villanova in Friuli, N.N.E. of Venice (Fig. 4). As the cross-sections show (Fig. 5), the underground river-channel is partly dissolved out of limestone but mainly eroded in underlying shales with thin beds of sandstone. The wide underground river-loop shown on Fig. 4 extends around the sides of a superficial valley which was excavated by the Rio Zafratta subsequently to the formation of the underground river-channel. The solution and excavation of the channel are assigned mainly to a period preceding the Glacial Period of North-West Europe, that is, to Pliocene times.[9] This notable example will serve to illustrate the dangers of leakage from reservoirs through ancient underground channels in limestone country.

The "swallow-holes" prevalent at the surface in limestone country result from the collapsed roofs of underground cavities (Fig. 6). A considerable area is often covered with these circular depressions, and wherever they are found, the question of leakage has to be considered. In some places, unfortunately, there may be no surface indication of danger, an especially unfortunate case being that of the Monte Jacques reservoir in Andalusia, situated in an area of Jurassic limestone. After the reservoir had partly filled, the water drained away. It was then found that a swallow-hole had formed in the alluvium-covered floor of the reservoir through which water had escaped down-valley. An arch dam 234 ft. in height spanned a gorge cut in the limestone, and leakage around the abutments through cavernous rock also contributed to the failure.[10]

It must be stated, however, that many instances are on record of water-tight reservoirs situated in limestone country, for example, in parts of the Venezian Alps. Even when cavernous, limestone should not be condemned without exploration. Downvalley leakage of water from a reservoir depends upon the extent to which the limestone has been cavitated by solution. At an early stage in the process the movement of water is transverse to the valley and towards the

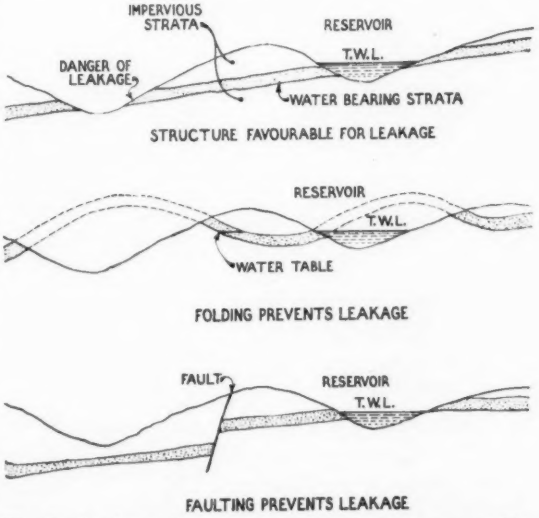


Fig. 8. Leakage from a reservoir through permeable strata (shown by stipple) depends upon the geological structure

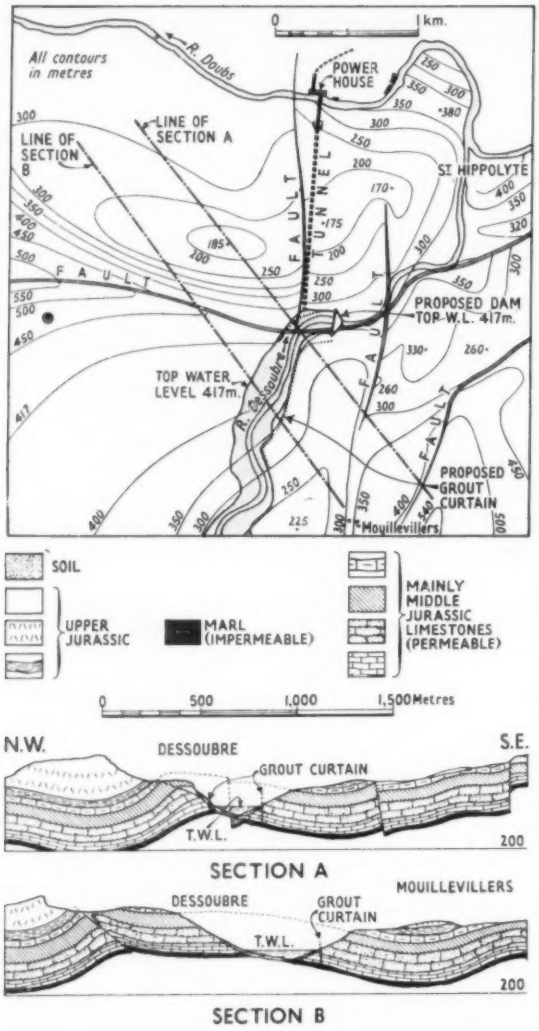


Fig. 9 (above). Sub-surface contouring of an impermeable bed (Lias marl) underlying permeable Jurassic limestones, in the geologically much folded French Jura, determines the lateral extent of the grout curtains required for making a proposed reservoir watertight; (below) Geological sections across the proposed reservoir site constructed from a map on a scale of 1 : 20,000. (After M. Lugeon, "Barrages et geologie," Rouge et cie, Lausanne, 1933)

river (Fig. 7); later, with more extensive cavitation, lateral channels are opened up, and a flow of water will then move laterally down the valley, causing serious leakage to take place.

Geological Structure and Leakage

Permeable strata may form the lining of a reservoir and yet, for structural reasons, no leakage may take place through them. If the strata are folded or faulted, impermeable beds may extend within the valley sides up to the top-water level of a reservoir. Diagrammatic sections on Fig. 8 show how structure of rocks may intervene either favourably or unfavourably in the matter. It has to be emphasised, however, that the conditions imposed by geological structure must be favourable *at all points* below top-water level, and it need hardly be added that to prove this requires prolonged research by a geologist, and probably exploration by borings or trial pits at any doubtful places he may locate. An especially instructive case is described below in some detail, and from its consideration the value to the engineer of detailed geological mapping will be apparent.

In the valley of the Dessoubre, a tributary of the River Doubs in the French Jura, east of Besánçon, permeable limestones of Middle Jurassic (Inferior Oolite) age cover most of the area.[10] They are underlain by marl, an impermeable stratum of Lias (Lower Jurassic) age. The marl is exposed at certain places along the river itself, both marl and overlying limestones being folded in a series of steep-sided anticlines and synclines. The subsurface levels of the marl were realised to be a main object for study. A detailed geological map of the area was therefore made by the late Professor Maurice Lugeon with the help of an assistant. A number of successive groups of limestone overlying the marl were identified on the ground, their various thicknesses estimated, and their outcrops drawn upon the map. From this detailed survey a number of horizontal (profile) sections were constructed across the valley at certain distances apart. Upon the accuracy of these sections the final stage of the investigation depended. The upper surface of the

Fig. 10. Fault, with broken rock (sandstone) and comminuted shale, traverses beds of lower carboniferous sandstone and shale, Calder Water, Midlothian, Scotland

impervious marl was next expressed on a map by contours. This final map is reproduced in Fig. 9, together with two of the sections from which the contours were obtained. Subsurface contouring of a particular stratum, such as a coal seam, is common geological practice, but in the case of civil engineering works it is seldom that a sufficient degree of accuracy is obtainable. In the Doubs valley case, however, Lugeon was able to demonstrate that the proposed reservoir could be made water-tight by the construction of grout curtains along part of the reservoir only, extending through limestone down to the impermeable marl along lengths a little higher on the valley sides than the proposed top-water level (Elevation 417 m.). Upvalley from the site of the dam a grout curtain of about 1,000 m. long was advised on the right side of the valley, and a shorter curtain about 400 m. long on the left side (Fig. 9).

Leakage Along Faults

The highly inclined belts of broken rock which normally accompany faults are well known water-carriers, and their outcrops are sometimes marked by pronounced green belts of vegetation or by lines of springs that issue from the fault-breccias. Faults rarely need be considered as a danger where the possible leakage paths are very long. On the other hand, a fault-zone outcropping across a divide at a point not far above top-water level of a reservoir may well entail special investigation and possibly the construction of a grout curtain or cut-off dam. Where a fault traverses argillaceous strata, the comminuted fault-fillings are usually water-tight as shown in the Scottish example in Fig. 10.

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[10b] pp. 26-28.

(To be continued)

Solar Energy

Some interesting comments on the utilisation of solar energy were made by Sir Harry Railing in his Presidential Address to the British Electrical Power Convention. Sir Harry said that our present sources of energy such as coal, oil and water power represented different types of stored energy derived from the sun, and likewise our food was derived from solar energy. Tracing back the ultimate source still further, we found that the sun's energy was itself due to thermonuclear reactions so that the world's power supplies, as well as the chemical energy that fed organic life, were all ultimately derived from nuclear energy. There was only one major source of energy, that which we were now releasing by nuclear fission of certain heavy elements such as uranium, which was not derived from the sun, but which was locked up in the atoms during the formation of the universe.

Of the possible methods for the conversion of solar to electrical energy the one needing least technological development was that by which solar radiation was used to raise steam and then to produce electrical power by conventional means. Heat was collected and focused on the boiler by a mirror system which had to be moved throughout the day to face the sun. Allowing for the power needed to keep an optical system facing the sun, it seemed unlikely that the overall efficiency of this method could ever be more than 5 per cent.

A second method was to utilise solar radiation to heat the hot junction of a thermopile, first thought of in the 1880's, and by 1913 efficiencies of only 0·01 per cent. had been reported, which was a long way from the theoretical efficiency of 15 to 20 per cent.

Attention had, therefore, been focused on a third method, the development of photo-voltaic cells of the barrier-layer type in place of thermo-junctions. Great impetus had been given to this development by the production of pure silicon and germanium, which were now the basis of many electronic devices. An array of barrier-layer voltaic cells exposed to the sun's radiation had an overall efficiency of 6 per cent., the theoretical upper limit being 20 per cent.

Theodor Bell & Co. Ltd., Kriens-Lucerne, celebrated recently the 100th anniversary of their foundation. We offer our heartiest congratulations to this well-known Swiss concern which has made such substantial contributions to the advance of water-power engineering at home and overseas.

VDI—Verein Deutscher Ingenieure. We have received the annual report of the Association for 1954 which, as usual, contains short accounts of the activities of the Directorate and district branches, as well as of the various special boards and committees. Since its revival in 1946, the number of members has increased from 15,267 to 29,454. Following the dissolution of the 20 district branches in Eastern Germany, membership is now restricted to Western Germany. The VDI-Press Office has also sent us a number of non-technical papers discussed at the Press Conference of May 5, 1955, in Bonn, as a preliminary to the Special Meeting held on May 17 and 18 in Münster.

Fig. 28. The Ångerman downstream of the power-station site, showing the new road bridge

Developments in the Ångerman Catchment

In this instalment we describe the Lasele development now in course of construction by the Swedish State Power Board. This will be the fourth State power station in the Ångerman catchment, the others being Kilforsen—already described in this series—Forsmo and Nämforsen.

PART THREE

AS explained in the first article in this series, Lasele will be the fourth station in the Ångerman catchment to be built by the Swedish State Power Board. The development lies on the Ångerman river about 8 km. above the Fjällsjö confluence and a short distance above the outfall of the Kilforsen tailrace tunnel. It was commenced in 1952 and is due for completion in 1957. It will have a capacity of 124 MW and should produce 610 million kWh in a normal year.

The catchment area above Lasele is 12,525 sq. km., and initially a storage of 815 million cu. m. will be available, but when Vojmsjön, Ransaren, Kultsiön and Malgomai—four lakes forming part of the headwaters of the Ångerman—are regulated, the storage capacity will reach 2,200 million cu. m.

At Lasele the Ångerman has a mean flow of 178 cu. m. per sec., a normal flood flow of 779 cu. m. per sec., and an exceptional flood flow of 1,450 cu. m. per sec. After full regulation the winter flow in a normal year will be 168 cu. m. per sec., and in a dry year 113 cu. m. per sec.

A rockfill dam will span the river, having a triple-opening concrete spillway section, log chute, and intake structure towards the right bank. Two main generating units will be installed in an underground machine hall, supplied by two concrete-and-steel lined penstock shafts extending vertically downwards from the intake structure. From the machine hall a combined surge chamber and tailrace tunnel will lead to a tailrace canal which will discharge into the river some 2,700 m. downstream of the dam.

Branching off from the machine hall will be an underground transformer hall, and, as at Kilforsen, a cable shaft will communicate between each transformer chamber and the surface. A vertical cross

section and a plan of the underground station are given in Figs. 28 and 29.

When we visited the site last year the civil-engineering construction was at a most interesting stage. Two of the spillway sections of the dam were nearing completion and the river was diverted through the space to be occupied by the third spillway section and the log chute. A cofferdam protected the right bank where shuttering was being erected for the intake structure. To the left, on the site of the earthfill section of the dam, the foundation for the impervious core had been cleared to bare rock and was being grouted. Belowground the access roads had been driven, the excavation for the machine hall had been nearly completed, the roof concreted, and the penstock shafts stoped through. Considerable progress had also been made with the transformer hall and the surge-chamber section of the tailrace tunnel and with the open-cut working on the tailrace canal. Downstream of the site, we found a particularly interesting feature in a road bridge which had been built by somewhat unusual methods, and carried the main traffic across the river.

The Dam

The rockfill portion of the dam, a cross section through which is given in Fig. 31, will have a crest length of 290 m., a crest width of 6 m., and will be 25 m. high to lowest foundation. It will require about 60,000 cu. m. of impervious earthfill, obtained from a borrow pit about 2½ km. away, and about 110,000 cu. m. of rock spoil from the machine hall and tailrace tunnel. Two rows of grout holes have been cut, the upstream row being 8 m. deep and spaced at intervals of 1·5 m., and the downstream row 5 m. deep and also spaced at 1½ m. intervals. Test bores have been put down in front of and behind the grout curtain.

To serve the concrete section of the dam, a crushing and a concrete-mixing plant have been erected on the right bank. Euclid trucks laden with rock spoil from the excavations discharge to the hopper of a jaw crusher which reduces the material to below 150 mm. The crushed rock is then conveyed to a

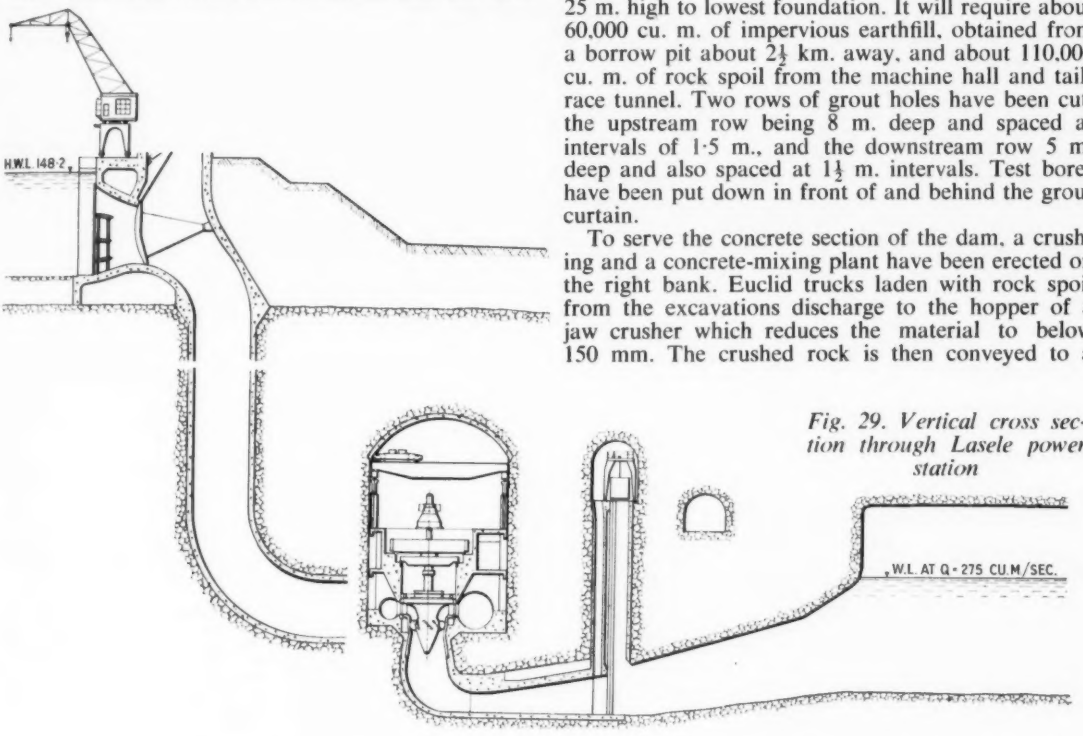

Fig. 29. Vertical cross section through Lasele power station

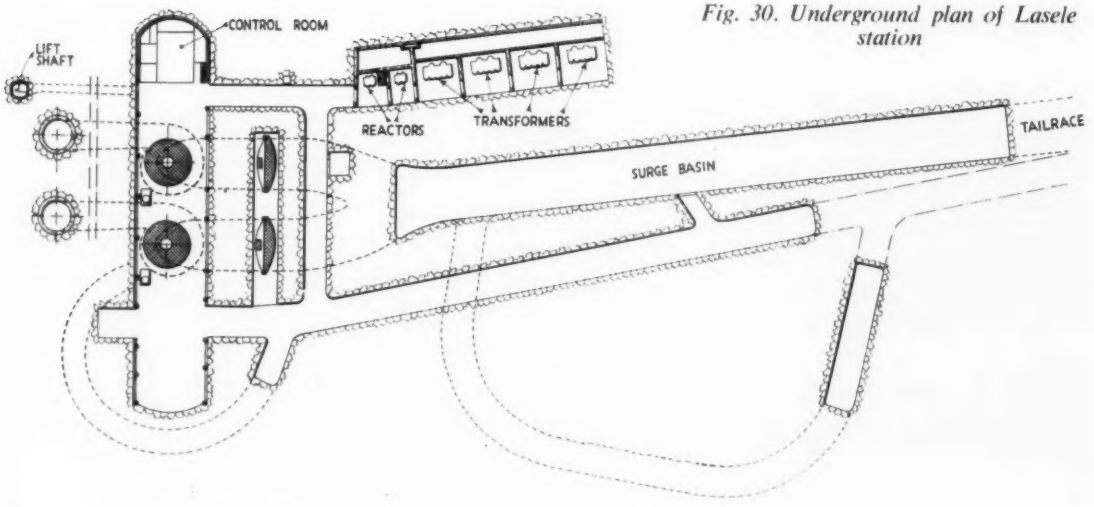

Fig. 30. Underground plan of Lasele station

screen where pieces under 64 mm. are separated and the pieces above 64 mm. are passed through a secondary crusher to reduce them to below 128 mm. These fractions are then elevated to the primary screens, where they are separated at 8, 32 and 64 mm. and fall into storage silos. The size under 8 mm. is not used.

The batching plant has hoppers for the three sizes made in the crushing plant as well as for two grades of sand (below 3 mm. and 3-8 mm.), obtained from borrow pits, and for cement. At the top of the batching plant a revolving chute distributes sized material to its appropriate hopper, and a screen is also available to deal with mixed material. The mix is weigh-batched and dropped to either of two Thornborg & Lundberg 1 cu. m. rotary mixers, which can discharge either to a bucket which is lifted over the dam by a cableway or to a lorry for road transport.

In frosty weather the concrete mix is heated by steam-heating coils supplied by an electric boiler, and by this means it is possible to mix and place concrete in temperatures down to −20°C.

Low-heat cement is used almost exclusively, and for a normal mix 310 kg. of cement is used per cubic metre of concrete, which weighs 2,250 kg. Trekoll air-entraining agent is added to give 3-5 per cent. aeration of the concrete. Full strength of the concrete is attained after 90 days. Continuous labora-

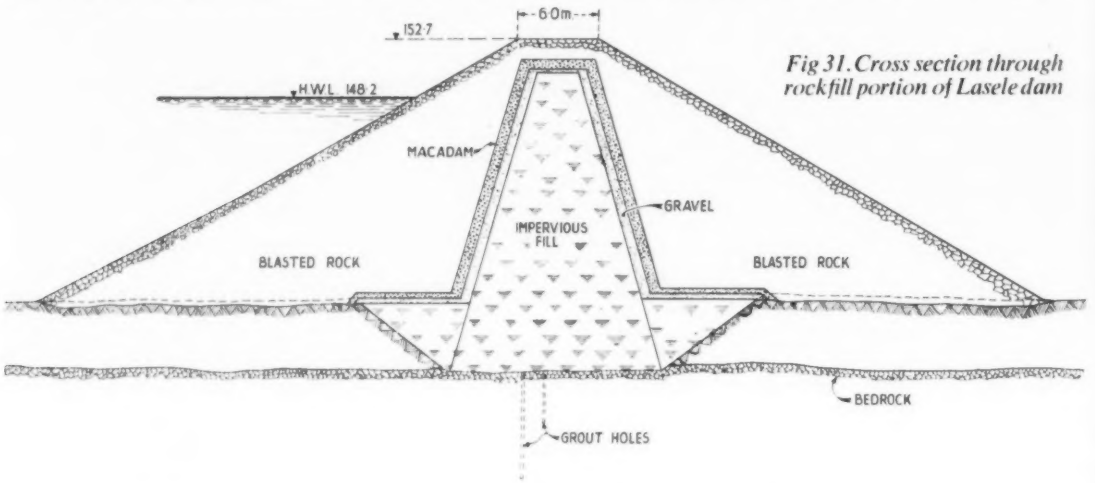

Fig 31. Cross section through rockfill portion of Lasele dam

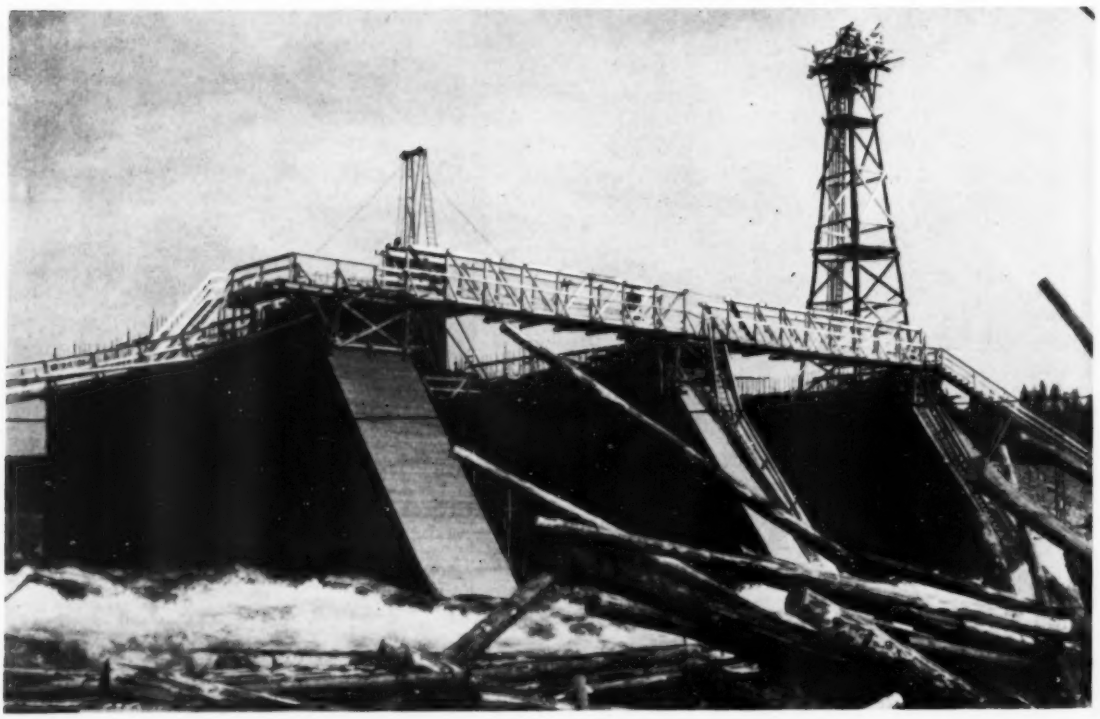

Fig. 32. Spillway portion of Lasele dam under construction

Fig. 33. View inside the tailrace surge chamber during excavation

tory control is exercised over the quality of the concrete, and test cubes of 20 cm. side have to show a minimum strength of 300 kg. per sq. cm. after seven days. Usually a strength of 400 kg. per sq. cm. is attained, and values up to 600 kg. per sq. cm. have been reached.

The batching plant can mix 170 cu. m. of concrete in an eight-hour shift, and has produced up to 5,360 cu. m. per month. On one section of the dam, where an uninterrupted run was possible, 2,200 cu. m. were placed in five days.

Fig. 32, taken at about the time of our visit, shows two of the spillway sections approaching completion, the river being diverted over the site of the third. All three openings will be 15 m. wide, and two of them, controlled by rising-type sector gates, will be 8·3 m. deep from high-water level to sill. The third opening will be controlled by a similar sector gate and will be 5 m. deep to sill. With all gates fully open the total discharge will be 1,698 cu. m. per sec.

Between the spillway section and the intake structure there will be a log-chute intake consisting of a floating structure 12 m. wide by 12 m. long leading to a chute 1,700 m. long. It is designed for a water flow of 10 cu. m. per sec. and a timber delivery of 20,000 logs per hour.

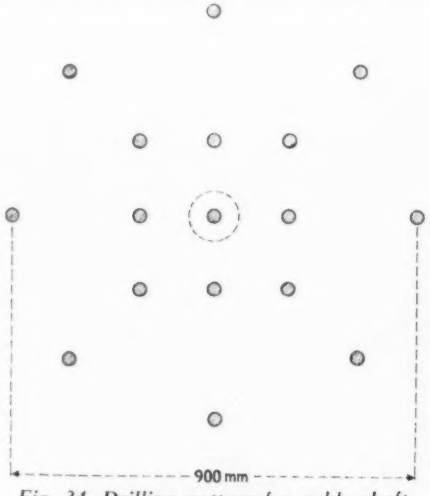

Fig. 34. Drilling pattern for cable shaft

Excavation Procedure

The station plan, Fig. 30, shows not only the permanent excavations but also some of the construction roads. All transport is handled through the permanent access tunnel, which reaches the machine hall at exciter-floor level. Branching left from the access tunnel a construction road loops under the access tunnel to the surge chamber, which it reaches at the floor level of the bottom drive. The upper part of the chamber is being carried out by a top drive, to which a short construction road leads right from the access tunnel. The middle part of the surge chamber has then to be stoped out. A view taken during the top drive appears in Fig. 33.

Farther down the main access tunnel a permanent transport cross road leads to a passage connecting the machine hall and transformer hall at exciter-floor level. From this passage, the transformer hall was opened out full face, but the method used for driving the cable shafts from the transformer chambers to the surface differed from that adopted at Kilforsen, where they were trepanned to full bore in one operation. At Lasele the full depth of each shaft was drilled—about 13 m.—according to the drilling pattern shown in Fig. 34. The central hole was then enlarged to 110 mm., as shown dotted, by a star-shaped bit having a central peg to register in the hole. This cutter operated from the bottom upwards, and an average advance of 5-6 cm. per min. was accomplished, although figures up to 8-10 cm. per min. were attained. Blasting was carried out from the bottom

upwards, and an average lift was achieved of 4½ m., the maximum being 5½ m.

Another construction tunnel (not shown in Fig. 30) was driven from the ground level to the roof of the machine hall which was opened out and concreted from this point before the main body of the machine hall was excavated. Access for excavating the machine-hall basement has been afforded by a further construction tunnel branching left from the main access tunnel a little way before it reaches the machine hall and looping round as seen in Fig. 30. This road is large enough to permit the passage of a North West 1·9 cu. m. shovel.

Fig. 35. Tailrace canal in course of excavation

Fig. 36. Half-arches of road bridge being lifted by derricks

Fig. 37. Half-arches pivoted over and being mated up

The ruling gradient on all roads is 1 in 7, and transport is effected by a fleet of 26 Euclid 13-ton trucks, of which up to 15 are in use at any one time. Rock removal has averaged 15,000 cu. m. per month.

About 100 Atlas Diesel RH67 rock drills with pneumatic pushers are in use for tunnelling, about ten RH65 drills with raising feeds for stoping, three BVB12 wagon drills for benching, and three BBD11LT light drills for breaking up boulders, drilling bolt holes, and other incidental jobs.

Compressed air is supplied from two stations. Tools in the machine hall and the central workings generally are supplied from a station comprising two Atlas water-cooled two-stage AR512 compressors each delivering 807 cu. ft. of free air per minute at 100 lb. per sq. in. and three AR5 compressors each delivering 1,000 cu. ft. of free air per minute at 100 lb. per sq. in. and having a maximum working pressure of 120 lb. per sq. in. For the tailrace tunnel and canal a second station is in service containing five Atlas water-cooled two-stage OK8 compressors each delivering 560 cu. ft. of free air per minute at a pressure of 100 lb. per sq. in.

A view of the tailrace canal in process of excavation by a 3 cu. m. Menck and Hambrode drag scraper appears in Fig. 35.

To ventilate the underground workings two electrically driven fans, each of 10 cu. m. per sec. capacity, are located at the tunnel mouth, feeding through a sheet-metal duct 1 m. in diameter; a third fan, of 5 cu. m. per sec. capacity, is placed inbye for local circulation to the tailrace tunnel roof through a 60 cm. duct, and a fourth fan also of 5 cu. m. per sec. capacity, serves the machine hall.

Permanent Road Bridge

Downstream of the power station is the permanent road bridge mentioned previously, which can be seen in Fig. 28. A bridge was needed in any case for the transport of materials to the dam site, but there was also a

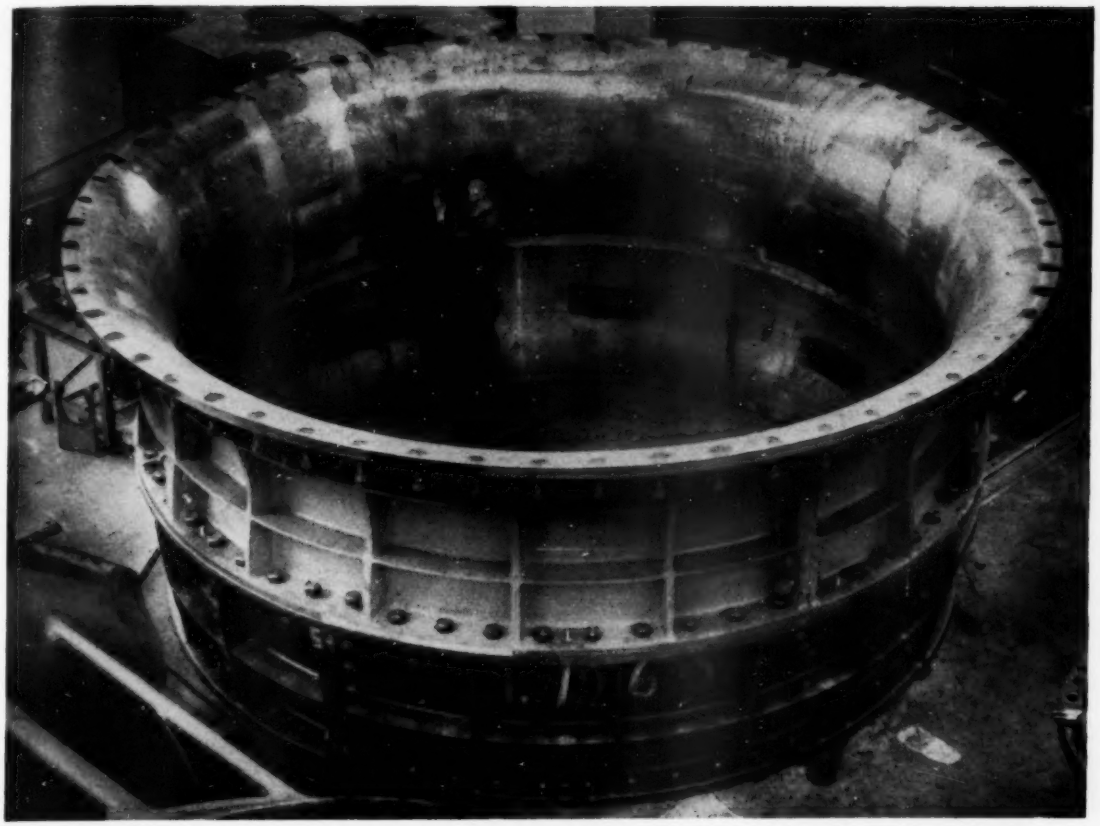

Fig. 38. Spherical discharge ring of one of the turbines in the works of Nydqvist & Holm A.B.

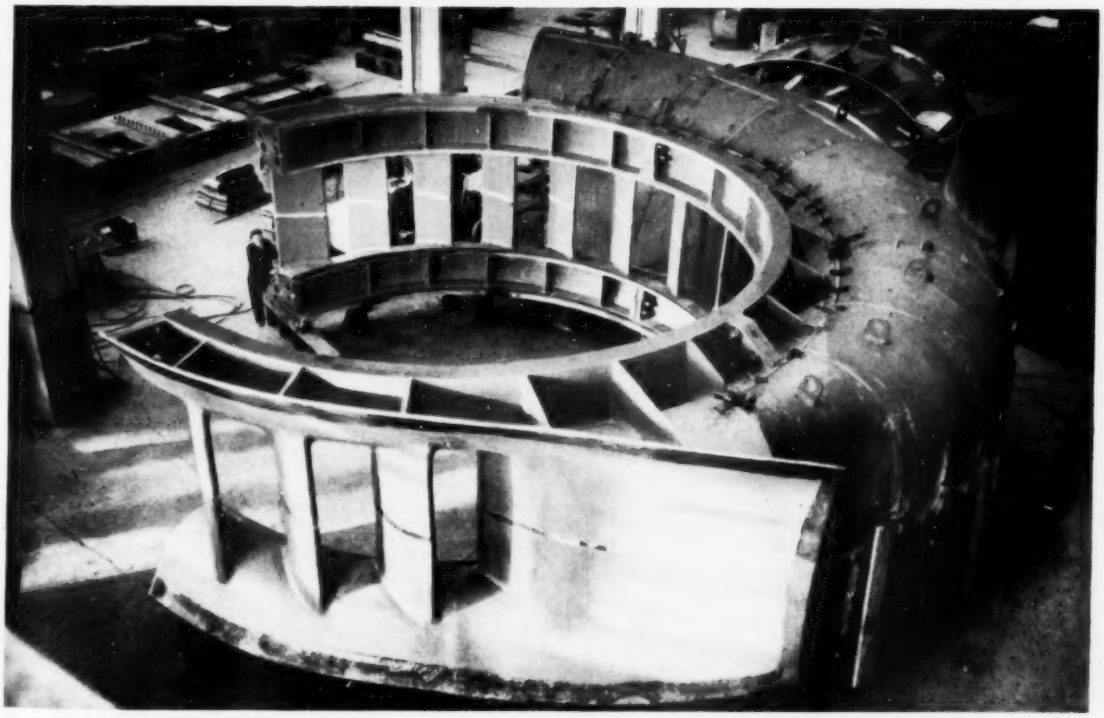

Fig. 39. Part of the stayring and spiral casing in course of assembly in the Nydqvist & Holm works

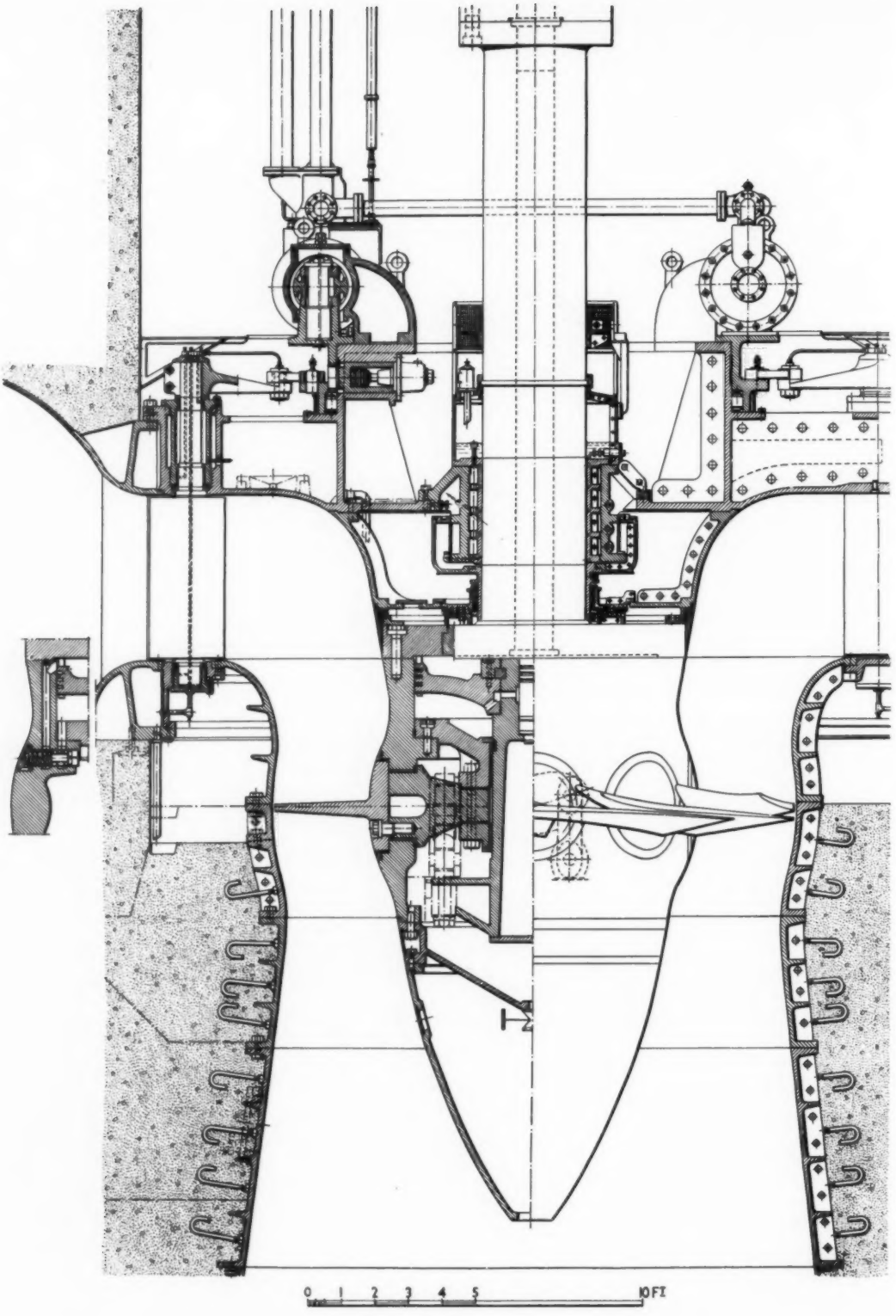

Fig. 40. Part sectional elevation through NOHAB 25 m. head 87,200 h.p. Kaplan turbine

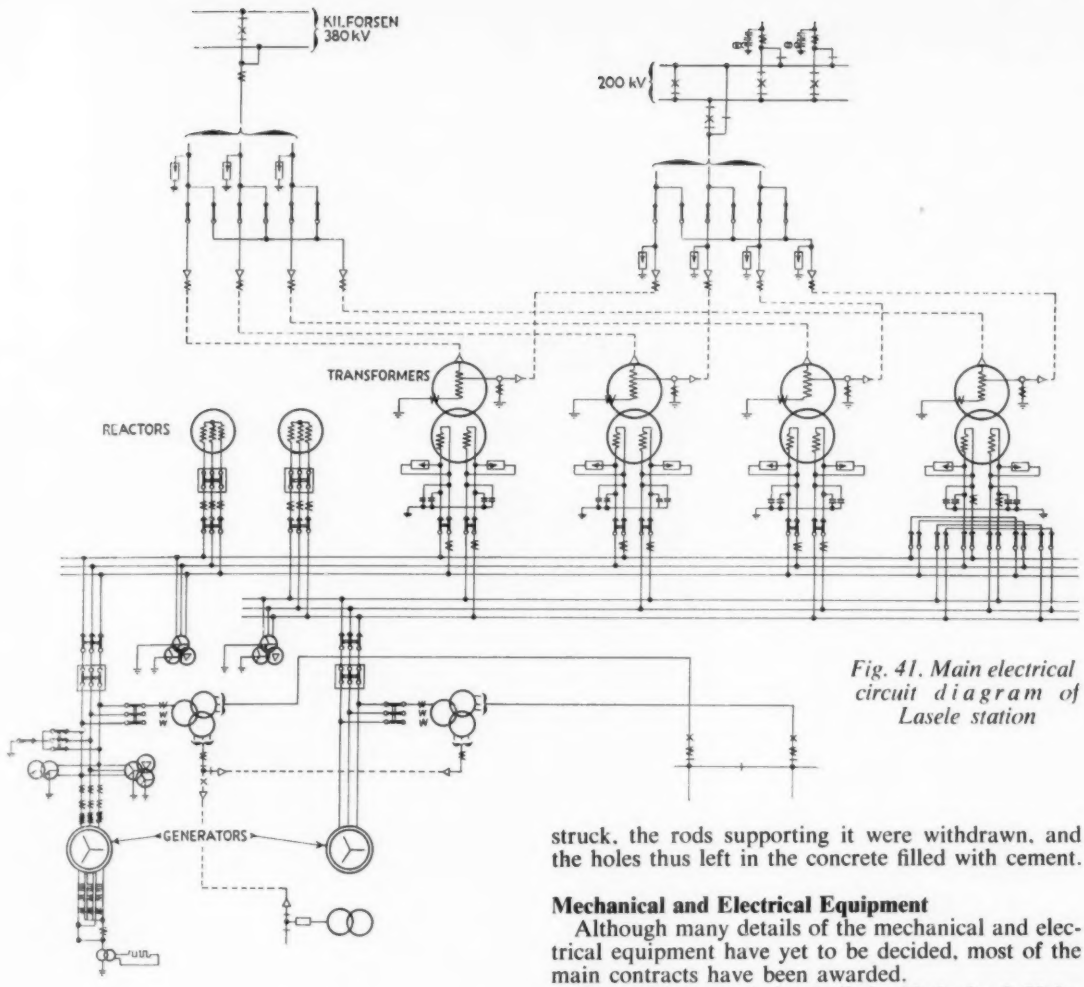

Fig. 41. Main electrical circuit diagram of Lasele station

demand for a permanent bridge for the convenience of the district, and the cost of such a bridge was therefore shared between the State Power Board and the Road Authority.

The bridge was designed by Mr. A. Cronholm of Kjessler & Mannerstral A.B., and consists of a fixed reinforced-concrete arch from which the roadway is suspended by steel rods, the arch trunnions being carried on reinforced-concrete abutments.

Considerable interest attaches to the method by which the arch was constructed. The core of the arch is formed by a light steel lattice framework, which was assembled in two halves on opposite sides of the river, each half, weighing 16 tons, being assembled on its back and with its foot engaging the abutment trunnion. A derrick, 26 m. high, was then erected behind each half-arch, and the two halves were then lifted, as seen in Fig. 36, pivoted over on their trunnions, as seen in Fig. 37, and finally brought together. Everything had been carried out so accurately that the two halves mated within one inch.

Steel reinforcement, bolts for the roadway suspension rods, and fixings for the shutterings were then welded to the lattice, the shuttering built up, and the concrete poured. After the shuttering had been struck, the rods supporting it were withdrawn, and the holes thus left in the concrete filled with cement.

Mechanical and Electrical Equipment

Although many details of the mechanical and electrical equipment have yet to be decided, most of the main contracts have been awarded.

The turbines are being built by Nydqvist & Holm A.B., and will be the highest-head Kaplan machines to be installed in Sweden. They will operate on a gross head of 52 m. and each unit will generate 87,200 h.p. The runner in each case will have eight blades and will work within a spherical-section discharge ring, as seen in the sectional drawing, Fig. 40. A view of the discharge ring in the makers' works appears in Fig. 38, and in Fig. 39 can be seen the stay-ring and spiral casing in course of shop assembly. The parts will be welded together on site.

Both sets will be controlled by ASEA/NOHAB electro-hydraulic governors.

The alternators, transformers and reactors will be of ASEA manufacture. Each alternator will be rated at 70 MVA at 0·9 power factor, normal speed 150 r.p.m., and will be designed to withstand a runaway speed of 300 r.p.m. They will generate at 13·5 kV, and the voltage regulators will be of the ASEA transductor type with impulse exciters.

A 60 MVA shunt reactor will be connected to the bars of each generator.

The transformer arrangement (see Fig. 41) will be identical in principle with that obtaining at Kilforsen, but there will be differences of detail. As there will be only two generators each single-phase trans-

former unit will have only two primary coils instead of three as at Kilforsen. A further difference arises because it is desired to use Lasele as a connecting link between the 200 kV system fed by Långbjörn and Nämforsen and the 380 kV grid, and for this purpose the secondary winding of each unit is of the auto-transformer type, the 200 kV and 380 kV systems being connected to the two limbs of the auto-transformer. The rating of the three-phase bank will be 97·5/97·5/450/585 MVA at 13·5/13·5/235/400 kV.

Details of the switchgear have not yet been decided, but as mentioned in our previous article, the Lasele supply will be handled by the switchyard at Kilforsen and the transformer high-voltage circuit breakers will be located there.

Camp Arrangements

We cannot conclude without a reference to the excellence of the living arrangements made at Kilforsen for staff and workmen at both Kilforsen and Lasele, for there is a sense of comfort that one would not expect to find in temporary barracks. In the unmarried quarters, for instance, of which Fig. 42 is typical, sleeping accommodation is arranged in two-bedded rooms instead of a large dormitory. Married workers have separate bungalows, such as seen in Fig. 43, and are encouraged to buy them on a deferred-payment system. As these houses are prefabricated they can be moved when the workman is transferred to another construction site. Houses for staff are similar but better appointed, and indeed appear to be as comfortable as permanent homes. A number of permanent houses have also been built.

Fig. 42. One of the huts for unmarried workers at Kilforsen

Fig. 43. Separate bungalow for married employee

Fig. 44. Community centre at Kilforsen

A community centre (Fig. 44) has been built at Kilforsen, as well as other service buildings such as a hospital, fire station, laundry, canteen, and so forth. Indeed, the wellbeing of the staff is obviously regarded as a matter of the utmost importance.

Acknowledgments

We are greatly indebted to the Swedish State Power Board for their permission to publish these articles on Kilforsen and Lasele, for the facilities they so kindly afforded us to visit these developments, and for the information and illustrations they have placed at our disposal. We are particularly grateful for the courtesy and assistance on site of Mr. S. Lindberg, the Resident Engineer, and of Mr. G. Andreen, one of the Civil Engineers. We have also received much valuable assistance from Atlas Diesel A.B., Allmänna Svenska Elektriska A.B., Karlstads Mekaniska Werkstad A.B., and Nydqvist & Holm A.B., all of whose works we visited when in Sweden.

Models of Hydraulic Structures

After illustrating the theoretical basis of scale models, the author discusses some of the practical problems encountered. Experiments carried out on three scale models for hydro-electric schemes are described.*

By DAVID H. KENT, M.A., A.M.I.C.E.†

PART ONE

THIS article was originally written with civil engineers in mind, but scale models of fluid-flow phenomena are of interest to engineers engaged on many other kinds of work. For instance, the designer of a Venturi Tube for some special purpose may find it impracticable to test or calibrate the full-size device, but a model of smaller size—or using a different fluid—can probably solve his problem. Pump and turbine manufacturers find that models provide an economical and accurate means of developing and testing new designs, and the extensive use of model tests by aeronautical engineers is well known. Civil engineers themselves use models to investigate the effects of new harbour and river works, to design sea walls, and for various other purposes.

One great advantage of small-scale models is that it is so much cheaper to alter the model than the prototype. Because of this, much more novel and possibly economic designs can be tried out than would otherwise be possible.

The measurement of velocities, heads and flows has to be carried out on the model just as it would on the prototype, and the smaller quantities are not necessarily easier to measure to the required percentage accuracy. However, to offset this, instruments and methods can be used in the laboratory which could not be used at the actual site.

A use of models which should not be overlooked is that of demonstrating a phenomenon. Something which can be seen is much more convincing to most people than a mathematical and theoretical proof, although in both cases the accuracy depends on correct assumptions.

The design of the waterways of a new hydraulic structure may be approached in three ways. The first is by pure reasoning and theory, the second is by experience of similar structures, and the third is by hydraulic model tests.

Simple structures, such as an overflow spillway with a normal ogee crest shape, or a relatively straight channel, can easily be designed from theory and experience. But as soon as complications are introduced, a model test is often the most efficient way of checking a design, since an exact precedent is rare and theoretical calculations become very lengthy and involved. In addition, because the failure of a hydraulic structure may cause loss of life—particularly in this and other relatively crowded countries—a model may be constructed and tested as a check on calculations.

It is essential that civil engineers should know the possibilities and limitations of a model so that they can decide when to ask for one, and so that they can appreciate the true significance of the results.

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Theoretical Basis: Laws of Similarity

The most certain way of finding the true equations for the operation of a model—hydraulic or otherwise—is by dimensional analysis. In each new problem we have but to decide which quantities can conceivably affect the flow, and the analysis gives us the relevant relationships. The basis of the method is the fact that any complete physical equation must be dimensionally homogeneous throughout; that is, every term in an equation, when reduced to basic dimensions of length, mass, and time, must contain identical powers of each of the respective dimensions (e.g., ML/T^2 in an equation of forces).

The dimensions of some of the quantities encountered in hydraulic model studies are given below, together with the symbols used in the text.

Quantity	*Symbol*	*Dimension*
Length, depth, etc	l, d, etc.	L
Area	A	L^2
Volume	V	L^3
Time	t	T
Velocity	v	L/T
Acceleration	a, g (gravity)	L/T^2
Discharge	Q	L^3/T
Kinematic viscosity	ν	L^2/T
Mass	m	M
Force	F, W (weight)	ML/T^2
Density	ρ	M/L^3
Dynamic viscosity	μ $(=\rho.\nu)$	M/LT
Surface tension	σ	M/T^2
Pressure intensity	p	M/LT^2
Elastic modulus (bulk)	K	M/LT^2
Energy and work	E	ML^2/T^2
Power	P	ML^2/T^3

The method of dimensional analysis can be found elsewhere [1-4]; here the intention is to derive only the relationships occurring frequently in hydraulic model work, and to use an approach which is not so generally useful but which is, perhaps, easier to follow in a particular case.

For complete similarity the requirement is that there shall be geometric, kinematic and dynamic similarity between prototype and model. The following ratios, prototype to model, thus appear:

Geometric (proportions)	$\frac{l_{p_1}}{l_{m_1}} = \frac{l_{p_2}}{l_{m_2}} = \ldots\ldots = l_r$	(length)
Kinematic (motions)	$\frac{t_{p_1}}{t_{m_1}} = \frac{t_{p_2}}{t_{m_2}} = \ldots\ldots = t_r$	(time)
Dynamic (forces)	$\frac{m_{p_1}}{m_{m_1}} = \frac{m_{p_2}}{m_{m_2}} = \ldots\ldots = m_r$	(mass)

$$\frac{F_{p_1}}{F_{m_1}} = \frac{F_{p_2}}{F_{m_2}} = \ldots\ldots = F_r \quad \text{(force)}$$

where l_p denotes a prototype length, represented in the model by a length l_m, etc.

These equations are connected by Newton's second law of motion:

$$F = m \,.\, a$$

so that we get: $F_r = \dfrac{F_p}{F_m} = \dfrac{m_p \,.\, a_p}{m_m \,.\, a_m} = \dfrac{m_p}{m_m} \cdot \dfrac{l_p/t_p^2}{l_m/t_m^2}$

$$\text{or } F_r = m_r \cdot \frac{l_r}{t_r^2}$$

Now $m \,.\, \dfrac{l}{t}$ is an inertia term which can be found for a given particle of fluid. But what forces can act on the particle?

In hydraulic model work there are four possible forces:—1, Gravitational; 2, Viscous friction; 3, Surface tension; 4, Elastic.

Thermal effects are important here only in that densities may be affected, but models involving transference of heat necessitate taking that into account in developing the appropriate model law.[5,6]

If *gravity* is the *only* force acting, then:

$$F = W = m \,.\, g$$

which leads to: $F_r = m_r \,.\, g_r$

but $F_r = m_r \cdot \dfrac{l_r}{t_r^2}$ (see above)

whence $t_r = \sqrt{\dfrac{l_r}{g_r}}$ (cf. the pendulum)

and $v_r = \dfrac{l_r}{t_r} = \sqrt{l_r \,.\, g_r}$

from which: $\dfrac{v_r}{\sqrt{l_r \,.\, g_r}} = 1$

$$\frac{v_m}{\sqrt{l_m \,.\, g_m}} = \frac{v_p}{\sqrt{l_p \,.\, g_p}} = \boldsymbol{F},$$

where $\boldsymbol{F}$ is the Froude number.

If the force acting is one *only* of the other three possible forces, we get, by similar reasoning:

Viscous friction only

$$F_r = \mu_r \,.\, A_r \,.\left(\frac{\mathrm{d}v}{\mathrm{d}d}\right)_r$$

$$F_r = m_r \cdot \frac{l_r}{t_r^2}$$

whence

$$t_r = \frac{\rho_r \,.\, l_r^2}{\mu_r}$$

$$v_r = \frac{\mu_r}{\rho_r \,.\, l_r}$$

$$\frac{\rho_m \,.\, l_m \,.\, v_m}{\mu_m} = \boldsymbol{R}$$

(Reynolds No.)

Surface tension only

$$F_r = \sigma_r \,.\, l_r$$

$$F_r = m_r \cdot \frac{l_r}{t_r^2}$$

whence

$$t_r = \sqrt{\frac{\rho_r \,.\, l_r^3}{\sigma_r}}$$

$$v_r = \sqrt{\frac{\sigma_r}{\rho_r \,.\, l_r}}$$

$$\frac{\rho_m \,.\, l_m \,.\, v_m^2}{\sigma_m} = \boldsymbol{W}$$

(Weber No.)

Elastic only

$$F_r = K_r \,.\, A_r$$

$$F_r = m_r \cdot \frac{l_r}{t_r^2}$$

whence

$$t_r = \sqrt{\frac{\rho_r \,.\, l_r}{K_r}}$$

$$v_r = \sqrt{\frac{K_r}{\rho_r}}$$

$$\frac{\sqrt{\rho_m} \,.\, v_m}{\sqrt{K_m}} = \boldsymbol{C}$$

(Cauchy No.)

or $\dfrac{v_m}{(v_s)_m} = \boldsymbol{M}$

(Mach No.), where $v_s = \sqrt{\dfrac{K}{\rho}}$ = velocity of sound.

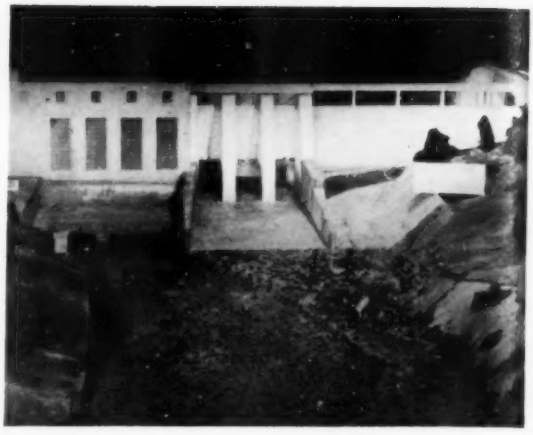

Fig. 1. 1 : 48 scale model of Torr-Achilty Dam and Power Station

In most models of hydraulic structures the fluid paths are so relatively short and the velocity changes so quick that the gravitational force predominates. (Viscous-friction forces can play a major part in enclosed flow, surface tension largely governs the action of bubbles and droplets, and elasticity is important when water hammer occurs.) In fact, it is better for one kind of force to predominate, for if two are present there are two equations instead of one and all the dimension ratios are interdependent. For instance, if *gravitational and viscous-friction* forces are present, we have:

$$t_r = \sqrt{\frac{l_r}{g_r}} \text{ and also: } t_r = \frac{\rho_r \,.\, l_r^2}{\mu_r}$$

so that $v_r = \dfrac{\mu_r}{\rho_r} = l_r^{\frac{3}{2}} \,.\, g_r^{\frac{1}{2}}$; and as $g_r = 1$ almost invariably, $v_r = l_r^{\frac{3}{2}}$,

and once we have chosen a model fluid the linear scale is fixed. If the protoype fluid is used (i.e., water usually), then $v_r = 1$, $\therefore l_r = 1$, and only a full-scale test can be expected to yield completely correct results!

The Reynolds, Weber and Cauchy laws are of use in connection with models of hydraulic structures mainly as an indication of possible errors, rather than as a basis for operation. With this guidance, it is, fortunately, often possible to arrange that the secondary forces are negligible and then use the simple Froude model law. For instance, glass, or a smooth plastic such as Perspex, on a model has a roughness approximately equivalent to that of concrete on a prototype some 20 times larger. However, it may often be used without appreciable error for scale ratios of 50 or so on models of dams and the like. This is, first, because the friction forces are small compared with the gravitational forces, and secondly because the "boundary layer" is thicker on the model. The thicker is this layer of slower-moving fluid, the less is the effect of the wall roughness on the flow; and so an exaggerated model roughness may be

masked. In general, however, there is some slight "scale effect," owing to the friction forces not being quite to scale. In spillway tests it usually means that the prototype has a better performance than the model, so that the model results are on the safe side.

Surface radii of curvature and minimum flow sections are kept as large as possible so that surface-tension forces on the model are not appreciable, for they never affect bulk flow on the prototype appreciably. However, where air bubbles or water droplets occur on the prototype, there is a big model scale effect. For instance, air is not as readily entrained on a model as on the prototype, and water does not break up into droplets as readily. This is referred to again later.

Thus, the basis on which models of hydraulic structures are usually designed is the Froude model law[7]. Once the length ratio has been fixed, after consideration of the various factors, the other ratios can be determined; i.e.:

$$v_r = \sqrt{l_r \cdot g_r} = l_r^{\frac{1}{2}} \text{ since } g_r = 1 \text{ usually}$$

$$Q_r = A_r \cdot v_r = l_r^2 \cdot \sqrt{l_r} = l_r^{\frac{5}{2}}$$

$$F_r = \rho_r \cdot g_r \cdot l_r^3 = l_r^3 \text{ if, for instance, water is used on model and prototype}$$

$$p_r = \frac{F_r}{A_r} = \frac{l_r^3}{l_r^2} = l_r \quad \text{,,} \quad \text{,,} \quad \text{,,}$$

$$E_r = F_r \cdot l_r = l_r^4 \quad \text{,,} \quad \text{,,} \quad \text{,,}$$

$$P_r = \frac{E_r}{t_r} = \frac{E_r}{\sqrt{l_r}} = l_r^{\frac{7}{2}} \quad \text{,,} \quad \text{,,} \quad \text{,,}$$

The length ratio chosen depends on the size and shape of the prototype, the accuracy required, the laboratory facilities, and the time and money available. However, as a rough guide, the smallest width of waterway should be over three inches and the smallest depth of flow over an inch, although these dimensions depend on the roughness, the curvature and so on.

This leads to normal model scales of from 1 : 20 to 1 : 80. A scale of $\frac{1}{4}$ in. to 1 foot or 1 : 48, for instance, is convenient with our duodecimal system of measurement; 1 : 50 would be used with the metric system. Sometimes a smaller model—say 1 : 96 or 1 : 100—is used as a pilot model to save time and expense on the main model. Models of coastworks or harbours must be excluded from this statement, for the extent of the prototype area is often so great that model ratios of 1 : 1000 or more are sometimes used.

Where a channel is very wide and shallow it is sometimes necessary to have a vertical scale different from the horizontal scale. This enables reasonable model water depths to be obtained without constructing a very large model. Thus, for instance, a river model in connection with the cooling-water circulating system for a steam power station had a vertical scale of 1 : 36 and a horizontal scale of 1 : 120. The pilot model had a horizontal scale of 1 : 360 and a vertical scale which could be adjusted by tilting the flume.

However, a vertical distortion is mainly confined to river and estuary models and is not usual where spillways, sluices and so on are being studied. This paper is confined to the latter; river and estuary models are discussed in an earlier paper.[8]

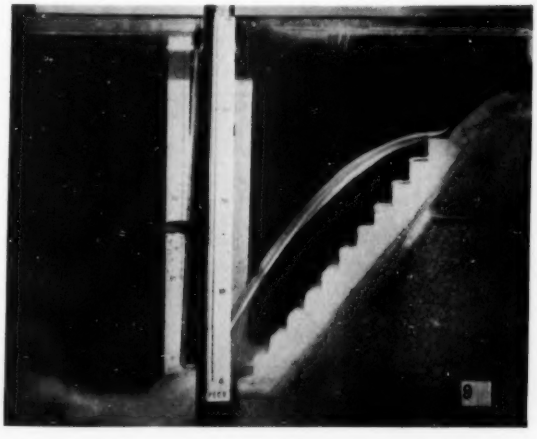

Fig. 2. 1 : 24 scale Torr-Achilty model, showing the tendency for the water to leave a stepped spillway face and so lose little energy as it descends

The Problems

Some of the problems which have to be investigated are:

(a) The calibration of spillways or sluices
(b) The determination of the best shape and size of channels and conduits
(c) The avoidance of low pressures or cavitation
(d) The limitation of air entrainment
(e) The limitation of river bed or bank scour and silting up or shoaling
(f) The prevention of excessive waves or surges

Most models are concerned with several of these facets, if not with most of them; but it is usually impossible to separate them out so simply. They can probably best be illustrated by describing a few actual tests, and the materials and methods used can be discussed at the same time.

Torr-Achilty Model[9]

The Torr-Achilty combined dam and power station, now under construction, is near Inverness, at the seaward end of the Conon basin scheme of the North of Scotland Hydro-Electric Board.

The structure, which is straight in plan, consists of four main portions:

(i) A non-overflow gravity dam on the south end
(ii) A Power Station with its tail-race channel situated mainly to the south of the river
(iii) A buttress dam located centrally in the river bed and embodying three large sluices which are the main provision for passing flood water
(iv) A gravity overflow-spillway, situated mainly on the north hillside, with a channel conveying the water back to the river.

Portion (i) was not involved in the hydraulic problems, and only the remaining three were modelled (Fig. 1), the work being done in the Hydraulic Laboratory of University College, Dundee. The scale of this main model was 1 : 48.

Problems of the types (a) to (e) were all involved in some measure.

(a) *Spillway Calibration and Design.* The design and calibration of the spillway were carried out with the aid of a 1 : 24 scale, two-dimensional model (Fig. 2). The model width of 9 in. thus represented an

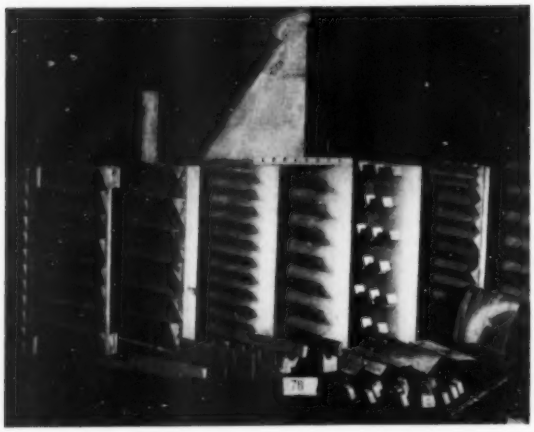

Fig. 3. Interchangeable face-pieces for 1 : 24 scale spillway model

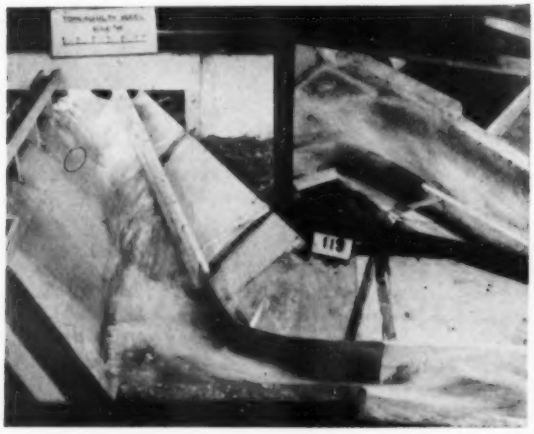

Fig. 4. Testing the spillway channel on the 1 : 48 scale model. Note the use of a mirror to give a complete picture

18 ft. length of spillway, and the model flow was $\frac{1}{24^3} = \frac{1}{2822}$ of the prototype flow over that length. Since the maximum spillway discharge is 5,000 cu. ft. per sec. over a 150 ft. length, a model flow of $\frac{2822}{5000} \times \frac{18}{150} = 0{\cdot}21$ cu. ft. per sec. was required. This could be supplied by the laboratory system. The depth of water over the spillway at normal maximum flood discharge was 1½ in., so that surface-tension forces were negligible here. Steps on the downstream face proved ineffective as a means of dissipating the energy of the falling water, because the top step was apt to act as a deflector and few of the other steps were then touched by the water. Various methods of leading the water on to the steps were tried, but it was found that a much better solution was to have a plain spillway face with splitter blocks projecting from it.

The dam itself was modelled in paraffin wax. This substance is very useful because it melts at about 130° F. and can therefore be cast nearly to shape; it is easy to carve and scrape to a given profile and has a glass-like smoothness. Difficulties, which can usually be overcome, are that it contracts considerably on setting, is easily dented or broken, and will float. In this case, the dam was simply gripped between the walls of the flume—and it only floated away once.

The various face-pieces and additional steps were made out of wax, wood or aluminium sheet (Fig. 3). Wood should not normally be used for models because of the extreme difficulty of preventing moisture absorption and the consequent swelling and distortion; also, in a glass-sided flume its expansion may crack the glass.

An essential part of the design of a model is to make it readily alterable. The construction may take longer in the first place, but ninety-nine times out of a hundred it saves far more time in the end. This cannot be emphasised too much, for there is a strong temptation to get results quickly and hope that the model will not have to be changed. In this particular case some eight different face-pieces were interchangeable by simply slackening off a couple of screws on the clamping block at the foot of the spillway.

Another important aspect of model work is recording the results. Sometimes it is convenient to compile tables, sometimes to plot graphs, but often time can be saved by photographing the flow and the data. On the Torr-Achilty models, over 200 photographs were taken; a plate camera was used and the plates developed immediately, so that any failure could be

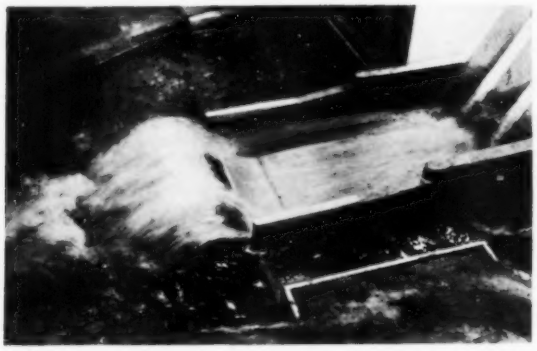

Fig. 5. Flow from the centre sluice with a curved "bucket" at the end of the apron

Fig. 6. The scour pattern resulting from 15 minutes of the flow shown in Fig. 5. The corner of the "bucket" can be seen on the right

Fig. 7. The model river bed during verification tests. The falls are nearly opposite the hut on the left

Fig. 8. The actual falls at Torr-Achilty on the River Conon, viewed from the hut shown in Fig. 7

rectified at once, ordinary cloakroom tickets being used for numbering. It is surprisingly hard to remember what one thought about a particular test after having witnessed several later modifications, and it is, therefore, essential to keep daily notes.

The shape and size of the channel carrying the spillway water back to the river bed was tested on the main, 1 : 48 scale, three-dimensional model. One test is shown in Fig. 4.

(b) *Vortices at the Sluice Intakes.* The tests on the three sluices showed that strong vortices were apt to form and draw air down with the water. It was found extremely hard to prevent vortices, but a floating grid did tend to break up the surface vortex and so keep the air out. In this case, the vortex is probably not very important, but at a turbine intake it must, of course, be avoided as far as possible. Vortices often seem to occur where there is "dead" water in which they can grow; the actual depth of the sluiceway appears to have little influence. Thus, the possibility of large vortices can be reduced by eliminating the dead water—either by altering the shape of the inlet or by drawing off the dead water in some way. It is not possible, however, to scale up vortex effects exactly from the model because, for instance, air bubbles are probably of the same order of size on model and prototype and therefore have the same order of rate of rise, since the fluids are the same[10]. It would seem though that models of this (1 : 48) or larger scale do give some indication of what will happen on the prototype. An accentuation of the model vortex may be obtained by, say, doubling the velocity of flow where this can be done, and this is often helpful in attempting to eliminate or reduce the vortex in the prototype.

(c) *Scour Tests.* The energy of a high-velocity sluice discharge has to be destroyed without endangering the structure. At Torr-Achilty, the rock downstream was known to be fissured at the surface, and so model tests were made to determine the best sill arrangement to prevent scour close to the structure at all flows. Fig. 5 shows flow with a curved bucket, and Fig. 6 shows the resulting scour pattern after 15 minutes' running. However, it was found that a lower, triangular sill was more effective, as well as simpler.[11]

The bed contours can be shown up by means of white string. If the water level can be controlled by a drain valve, the string can be set round the water edge each time the level is lowered, say 5 prototype feet. The results are only comparative, for the resistance of the prototype rock is unknown, but they are of great assistance to the designer, particularly as the effects of the downstream water level and every combination of sluice-gate openings can be tried.

It is suggested, however, that, unlike the example shown, one should avoid having the floor of the tank uncovered by the scour, as it may alter the pattern of flow somewhat.

The size of the model erodible material is not vital, within certain limits,[12] as there is no calculable time scale for these tests. Pea gravel, $\frac{3}{8}$ to $\frac{3}{16}$ in. nominal size, proved convenient, in that the rate of scour was

Fig. 9. Flow through a 1 : 4 scale round-bar screen, compared with the same flow through a flat-bar screen (above)

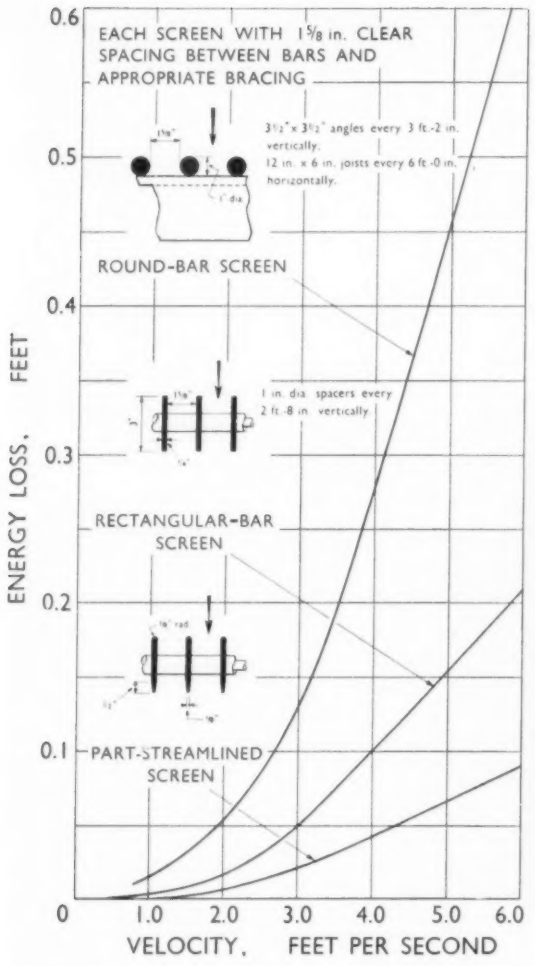

Fig. 10. Comparison of head losses caused by tailrace fish-screens, as estimated from $\frac{1}{4}$, $\frac{1}{8}$ and $\frac{1}{16}$ scale tests

reasonably fast but the stones were not then washed too far down the river. It is interesting to note, however, that in Switzerland they have records of the erosion of a certain bridge foundation over many years, so that for certain model tests the erodible material can be calibrated by being used on a model of that stretch of river.

When river and estuary models are run, the first step is to verify the model performance with any known performance data of the prototype. As far as models of hydraulic structures are concerned, the requirement is that the downstream stage discharge curve should be correct. Fig. 7 shows the River Conon during the tests before the dam structure was superimposed. The bed roughness and shape, and the outlet-gate settings were adjusted to give water levels in agreement with those recorded on site.

Where natural topography is to be included in a model, a visit to the site is of tremendous value to the model engineer. Photographs are sometimes a necessary substitute, though they are apt to be inadequate. Fig. 8 shows the falls at the actual Torr-Achilty dam site.

(d) *Head Loss at Fish Screens*[13]. Determination of the head lost at the proposed tail-race fish screens for Torr-Achilty involved some less-usual tests, particularly as an attempt was made to check the resulting force on the screens. Fig. 9 shows the loss at full flow across a round-bar-type screen, compared with that across a flat-bar-type screen with the same clear spacing. Fig. 10 gives a comparison of the estimated prototype losses at various velocities of flow for round-bar, flat-bar and streamline-bar screens, each with the same clear spacing and with the necessary bracing members. For a low-head station, the head saved with the better screen may be as much as 2 per cent. of the head on the machines, which is well worth consideration. However, recent tests of electric fish-screens may lead to a solution of this problem.[14]

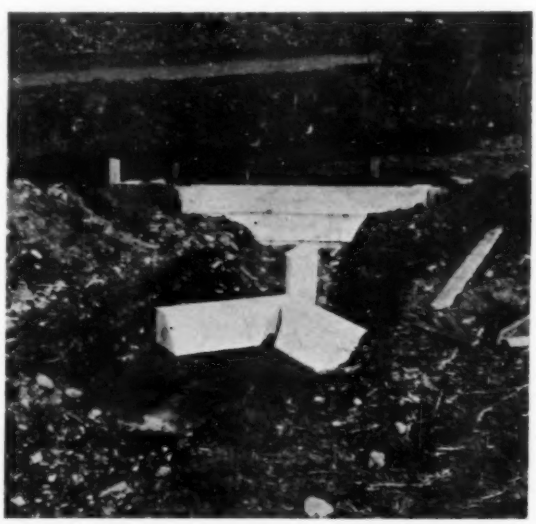

Fig. 11. A model diversion weir under construction outdoors, beside a stream

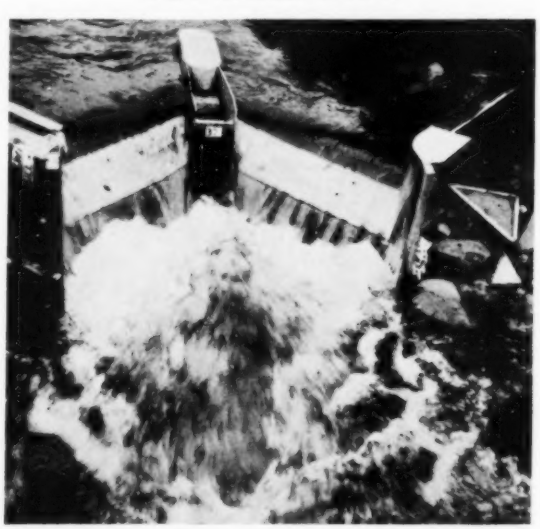

Fig. 12. Maximum flow over the 1 : 24 scale model of the V-shaped diversion weir

It is perhaps worth mentioning that useful information can also be obtained from model studies carried out in the field; the accuracy may not be up to laboratory standards but can be good enough for many purposes. Fig. 11 shows a model under construction beside a stream quite near the actual site in Scotland. "Catastrophic" flood flow over the V-shaped weir is shown in Fig. 12.

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(To be continued)

Institution of Civil Engineers' Conversazione

The exhibits at the annual conversazione of the Institution of Civil Engineers included, as usual, a number of great interest to engineers engaged on hydro-electric development. Messrs. Babtie, Shaw & Morton, of Glasgow, displayed a model of the first prestressed concrete dam to be undertaken in Western Europe. This dam, which was illustrated on page 262 of our July issue, is now in course of construction for the North of Scotland Hydro-Electric Board at Allt-na-Lairige in Argyllshire, and will be 1,360 ft. long, will have a maximum height of 83 ft. above the cut-off trench, and is being built in blocks 42 ft. long. Prestressing anchorages will be at 21 ft. centres, and each will consist of 28 high-tensile steel rods $\frac{1}{8}$ in. in diameter, concreted at the bottom into deep pits cut in the foundation rock. Each anchorage will be stressed by jacks to produce a load of 1,175 tons, giving a total anchorage load throughout the dam of 54,000 tons. This construction will save about 40 per cent. in the volume of concrete required, and will counteract tensile stresses on the dam face, ensuring a more watertight structure.

A fascinating study of helical motion in a curved channel is being made by Mr. T. M. Prus-Chacinski, A.M.I.C.E., under the guidance of Professor C. M. White, at the Hawksley Hydraulics Laboratory of Imperial College, London. This study is of great potential importance as it suggests means whereby silt transport and bed erosion at the bends of rivers and canals may be controlled. Various phenomena were demonstrated—some possibly for the first time—by a Perspex model consisting of a rectangular channel 6 in. wide and having two opposite-handed bends, 39 in. radius and 45° deflection, separated by a straight section 18 in. long. The presence and the character of the helix were effectively demonstrated by dyes, and the helix was shown to cause great variation in the rate of silt transport and to govern the pattern of erosion and deposition. Important findings were that the helical pattern is very sensitive to cross currents upstream, and that the helical component, once developed, dissipates very slowly and so persists for long distances. Thus it was shown that the helical motion in the first bend in the model greatly reduced that in the second, and that in its turn the helical component in the first bend could be controlled, and even eliminated, by vanes placed in the straight channel a considerable distance upstream. This suggests the possibility of overcoming bed erosion by upstream works without having recourse to expensive groins at the bend. Other experiments suggested the possibility of keeping silt clear of an off-take by similar methods.

The importance of soil mechanics to the study of dam sites is becoming increasingly recognised, and an exhibit by Mr. H. Q. Golden, D.Eng., A.M.I.C.E., of Soil Mechanics Limited, included a triaxial testing machine for soils and soft rocks in which lateral pressures up to 1,000 lb. per sq. in. can be developed. The ability to make tests with such high lateral pressures is important for rocks where the measured compressive strength is critical by comparison with the stresses that would be applied to the rock. A laboratory vane tester was also demonstrated enabling the shear strength of sensitive soils to be determined without extruding the samples from the original sampling tubes.

The Hydraulics Research Station of the Department of Scientific and Industrial Research, under Sir Claude Inglis, showed a model tide recorder for use with tidal and river models, particularly where measurements are required in narrow channels. A motor-driven probe follows the water surface, the input to the motor being the amplified output of a Wheatstone bridge one arm of which is the impedance between the probe tip and a submerged electrode. The motor also drives the slider of a potentiometer which is balanced against a second potentiometer driven by the motor of the recording pen. An accuracy of at least $\pm$ 0·015 in. for a 3 in. range is claimed.

Amongst a wide range of exhibits from Cambridge University mention should be made of a model, shown by Professor Hawthorne, showing the scouring of a river bed at the base of a pier, and of a twin-mirror extensometer, shown by Dr. R. D. Davies, for concrete creep tests.

The Fifth International Congress on Large Dams

The Fifth International Congress on Large Dams, which is a body associated with the World Power Conference, held its meeting in Paris from the 31st May to the 4th June last. Before the Congress opened, the French Committee on Soil Mechanics held a three-day meeting.

PART TWO

TO conclude our summary of the Papers on the economics and safety of different types of concrete dams (Question 17), the three Papers from Great Britain dealt mainly with experience gained in the Scottish hydro-electric developments. The first of these, by Mr. C. M. Roberts, M.I.C.E., of Sir William Halcrow and Partners, described the problems involved in heightening a gravity dam. The various methods which could be adopted were classified under three headings. The first consisted of the provision of vertical prestressed rods or cables passing through the dam near the upstream face and anchored in the rock foundation, and extending either up to the crest of the original dam or up to the crest of the raised portion. Secondly, there was the possibility of providing a concrete or stone-masonry slab or buttress resting against the downstream face of the original dam, but not adhering to it and merely pressing on it through a friction contact. Finally, there was the method involving the provision of a concrete or stone masonry slab or buttresses, monolithic with the original dam, which would thus be thickened to have exactly the same profile as if the dam had been built for the higher water level in the first place. The Mullardoch dam was used as a typical project to illustrate the problems involved.

Messrs. E. J. K. Chapman, B.Sc., M.I.C.E., M.A.S.C.E., and D. Fraser Campbell, B.Sc., A.M.I.C.E., of James Williamson & Partners, of Glasgow, presented a Paper describing recent developments in regard to massive buttress dams in Scotland and indicated the main factors influencing the design of these dams to suit the prevailing conditions and to produce the most economical results. An analysis of quantities and costs was given for three dams of the same type as Loch Sloy, and was shown how economy had lately been achieved by modifying the original design.

The final Paper in Question 17 was presented by Mr. James A. Banks, O.B.E., M.I.C.E., of Babtie, Shaw and Morton, and was of especial interest since it dealt with the prestressed concrete design employed by the North of Scotland Hydro-electric Board for the Allt-na-Lairige scheme. A uniform prestressed load was adopted throughout the length of the dam, which has a maximum height of 73 ft. above bed-rock level and an overall length of 1,360 ft. The dam is designed to afford a storage of 130 million cu. ft. of water and will be completed towards the end of this year.

The Lee-McCall system of prestressing, using Macalloy rolled high-tensile bars and couplers, was chosen in preference to the use of steel wire in cable form and the Paper sets out the reasons for this choice in full detail. Full scale tests were carried out on site to determine the anchorage value of the rock, and tests were also made on the wrappings used to prevent a bond between the bars and the concrete when prestressing. The "Denso" type of wrapping was adopted, this being a proprietary grease-type plastic, applied in the form of a paste as a primer, wrapped with tape impregnated with the same material, the final covering being thin bitumen sheeting.

Settlement of Dams due to Compressibility of the Dam Material or of the Foundation Soil, including Earthquake Problems (Question 18)

There was a strong contribution from the United States and from France to this section of the Conference.

Mr. K. S. Lane, Chief Engineer of the Soils and Geology Branch, Garrison District, Corps of Engineers, presented a Paper describing the methods used in designing for foundation movements at Garrison Dam. Here the major structures are founded on clay shale, which is essentially a formation of pre-consolidated hard clay containing pervious beds of lignite coal. Mr. Lane stated that in this project, which had been under construction since 1946 and was now approaching completion, rebounds of up to 2 ft. had already been measured as a result of unloading, due to the large excavations, and had been followed by settlements of a few inches as loads were added by the structures. It was found necessary to utilise articulated structures, each generally consisting of a series of rigid blocks with the intervening joints designed to accommodate more than the computed movements. The 260 ft. high intake structure was started as an eight-block structure and then converted by closure pours to its final four-block condition after part of the initial movement had occurred. The 29 ft. diameter tunnels and penstocks utilised numerous articulated joints, and at the power house the movement problem was a major influence in selecting the layers.

The behaviour of a rolled-earth dam constructed on a compressible foundation was the subject of the Paper from Mr. W. W. Daehn, of the Bureau of Reclamation, Denver, Colorado. He described the Fresno Dam, which was designed to suit a site in which the natural foundation consisted of silts and fine sands with pockets of clay below the 30 ft. depth, while there was also an abundance of suitable material for the construction of the impervious zone of the embankment. No pervious material was available for filter zones and there was a scarcity of suitable rock for providing a sufficiency of rockfill. The embankment was, therefore, designed to incorporate a large section

of impervious fill with relatively flat slopes. The designed crest of the embankment was only 77 ft. above the stream bed, and after 14 years, the foundation settlement now exceeded 8 ft. The dam had had to be recapped twice, but otherwise no adverse affects had resulted from such a high degree of settlement and the seepage was very small. It was thought that although settlement would continue to recur, its rate would be reduced with time.

Messrs. E. A. Woodhead, R. A. Sutherland and W. L. Wells presented a Paper dealing with the C. J. Strike Dam on the Snake River in Idaho, which consists of an earth dam, a concrete spillway, an intake and powerhouse, and two irrigation intakes. The head developed is 90 ft., but the maximum height of the earth dam is 132 ft. and the height of the spillway structure from the bottom of the base slab to the floor of the operating bridge is 115 ft. The dam is 3,220 ft. in length, and the power plant has a capacity of 82,500 kW. The dam is mainly founded on compact clayey silt, and was completed in 1952. The settlement of the spillway was only about one-third of the amount predicted, as it only settled 0·1 ft. during construction, an additional 0·06 ft. during the filling of the reservoir, and a further 0·04 ft. in the succeeding 18 months.

Two engineers from the Tennessee Valley Authority, Messrs. C. E. Blee and A. A. Meyer, described the measurement of settlements at certain dams on the TVA system and gave assumptions for earthquake loadings. Very complete figures were given for the settlements of a number of dams of different types of construction, and these will form a useful guide to engineers engaged on similar projects. The earthquake loading assumptions are of special interest, as in the lower section of the river from mile point 60 to the mouth there is a floor of old hard rocks which have many vertical faults. The rocks have moved on these faults in block fashion up and down, some very large displacements having taken place, while light shocks are frequent occurrences. The earthquake acceleration applicable to the design of rigid structures founded on rock was taken as 0·05*g*, with this acceleration acting either parallel to or normal to the axis of the dam. A series of shaking-table tests were carried out at Stamford University, and as a result considerable modifications to the original design of various dams were made, and these are detailed in the paper. In some cases these modifications led to an increase of about 35 per cent. in the volume of the retaining walls.

The final paper from the U.S.A. was by Mr. E. E. Esmiol, of the Bureau of Reclamation, who described the foundation consolidation work carried out beneath four of the Bureau's rolled-earth dams, as determined from field observations. The Paper described how the observations were made and how the vertical consolidation stresses in the foundations were computed according to the elastic theory of stress distribution. Natural compression curves developed from laboratory compression curves agreed to within 1 per cent. of the field compression curves. At 100 lb. per sq. in. the maximum observed field compression of the four dam foundations was 18·5 per cent. and the minimum observed field compression was 0·43 per cent.

An engineer who is specially well known for his contributions to civil engineering progress, Monsieur S. Ferry, of Electricité de France, contributed a Paper on ascultation data on the elastic or irreversible movement of dams under the effect of loads. A number of measurements are detailed in the Paper and in all cases a rotation of the rocky base of the structure was noted, amounting to about one-ten thousandth of a radian, thus showing the deformation of the foundation. The effects of the variations of temperature and weight of water having been eliminated, the data presented give a clear indication of the continuous permanent deformation of certain dams at certain periods after they had been completed. Most of the measurements were made by the aid of pendulums.

A Paper by Messieurs Blanchet and Neviere dealt with another aspect of movement in civil engineering structures—that of the settlement in the banks of reservoirs in the neighbourhood of the dams. Three dam sites were chosen for an extensive series of measurements, and the Paper devotes much thought to the best method by which these measurements may be carried out, both operationally and from the point of view of later computation. The relationship of the perceptible contractions and dilations of the banks of valleys, referred to load fluctuations, are discussed, and it appears that some fairly constant factors are revealed.

Two further Papers from France (the Donzère Dam and its foundations by Monsieur Henry and the settlements of the foundation of the Bort Dam by Monsieur Jeanpierre) relate to particularly interesting projects in France which have recently been carried out. Monsieur Henry explained in detail the instrumentation used to measure the settlements of the Donzère Dam, and Monsieur Jeanpierre, after mentioning that the Bort Dam is built on two very different kinds of rocks separated by a fault plane filled with greatly decomposed material, showed that the extensive sounding equipment installed had revealed that although the settlements occurred as anticipated, they were much less in magnitude than expected.

From Algeria came two papers; the first, by Monsieur Jean Salva, dealt with the interesting Sarno dam which is of a flexible nature, consisting of an earth dyke 30 m. in height and 310 m. in length at the crest. It is the first structure of this type in North Africa, and for this, and other reasons, great care was taken in measuring the deformations, many datum points being established. Some of these were inaccessible points within the foundations and in the puddling of the dyke; a *teleniveau* (remote indication device) was installed, having been developed to a very high degree of precision. The dam was completed in April, 1954, and although interesting figures have already been obtained it will not be until later, when measurements over a period of years will be available, that the full significance of this extensive measurement equipment can be determined.

The second Algerian Paper, by Messrs. Jean Guilhamon and Henri Castelnau, dealt with the Iril Emda Damon and Oued Agrioun, which is an 80 m. high embankment of small rockfill with a centrally-placed reinforced concrete structure for flood discharge. The compacting of the rockfill was of special interest, and among the most valuable parts of this Paper are those which relate to the methods adopted for measuring the effect of various types of compacting machinery.

The Ben-Metir Dam in Tunisia is a buttress dam founded on compressible soil overlying alternating layers of sandstone and clay, which were relatively

hard lower down, but were broken up near the surface. A very complete network of drainage pipes and galleries designed to avoid uplift and to accelerate the consolidation of the clay foundation were found necessary, and are described in detail in the Paper.

The British contribution to this section of the Conference took the form of two Papers. One related to the Hollowell Dam at Northampton, which is an earth bank with a puddle-clay-filled cutoff trench and a puddle-clay core wall, the wall being flanked on each side with selected retentive material. When the construction of the dam was almost completed, following a week of wet weather, which occurred after a long dry period, it was found that the puddle core wall settled and the downstream slope moved outwards, causing upheaval of the downstream stone toe and movement of the main and lateral drains. Soft clay found beneath the surface was removed and replaced with concrete, while sheet steel piling was driven through the bank down to the underlying rock band, and this had the effect of preventing further outward movement.

Although the Paper by Messrs. J. K. Hunter and H. G. Keefe, of Sir Alexander Gibb and Partners, emanated from Britain, it dealt with the design of dams and other engineering works for employment in active volcanic country. Among the points made by the authors was the fact that dams can be specially designed for such situations with complete success, as proved by the Cogoti Dam in Chile, which was severely tested in the earthquake of 1943, only five years after it had been completed. This rockfill dam was based on the following principles; the selection of side slopes which would not flatten under earthquake shocks, and the use of a watertight upstream face which could bend without rupture. After the earthquake, the leakage was practically unchanged. The authors also stated that tunnels constructed in rock appeared to be highly immune to earthquake damage, which only occurs at the portals. This led to the suggestion that, where practicable, underground arrangements should be adopted for hydro-electric works constructed in seismic regions. Also, the authors considered at length the methods of sealing cutoff walls to allow for earthquake conditions.

Two dams in Portugal were described in the Papers by Senhores Armando da Palma Carlos and L. H. Gomes Fernandes and A. A. Correia de Sousa. The Salazar dam is a rockfill structure 56 m. in height and its construction is described in detail. The settlements observed showed that on the downstream side there was more settling than on the upstream side, a fact which may lead to reconsideration in the future of the methods of hand-placing the rockfill.

The other Paper described the behaviour of the Salamonde dam during the first 15 months after filling. This is a thin arch concrete dam which was fully described by Senhor Fernandes in the November and December issues of WATER POWER in 1954. The small displacements which occurred after filling were measured by the geodetic method and these measurements are set out in the Paper in detail, with some interesting diagrams relating them to the different levels of the dam.

A further Portuguese Paper by Senhores Manuel Rocha, J. Laginha Serafim, A. Ferreira da Silveira and J. M. Ressurreicao Neto discussed the deformability of foundations rocks and described tests carried out by the Laboratório Nacional de Engenharia Civil. These tests related to the well-known Portuguese projects of Castel do Bode, Vila Nova, Cabril, Salamonde and Caniçada. The Laboratory tests are of great interest and the Paper also discussed major tests carried out on site. The Paper will therefore be of special interest to those engaged in this work, since very few comparable tests have been made. The use of strain gauges in the foundation rock is a technique capable of much extension, and is described in the Paper.

From Switzerland two Papers dealt with general questions of settlement, and a further contribution related to a specific project, the Rossens arch dam. Monsieur Henri Gicot described this dam, which was erected on soft sandstone having a marked tendency to elastic and plastic deformation, and then compared the measured deformations after the dam was commissioned in 1948, with the calculated figures.

Monsieu Henri Juillard discussed the possibility of settlements due to compressibility of concrete,and compared this with compressibility due to the foundation movement. He stated that when concrete reaches its normal temperature it behaves like an elastic material, and the experience gained from measurements for more than 20 years is stated to justify the principle of hyperstatic calculation when considering this aspect of dam design.

Messrs. A. Priessman and B. Gilg carried their Paper outside the dam itself to the elastic spreading of the abutments, and considered this feature in regard to the calculation of the spreading taking place in an infinite symmetrical V-shaped canyon under reservoir water load, for various steepness angles of the slopes.

Herr H. Möhle, of the German Federal Republic, dealt with the settling and displacements of the earth dam at Bever, which has a maximum height of 42·6 m. and was constructed between 1935 and 1938 by the use of some 1,000,000 cu. m. of different kinds of soil. The displacement after 15 years has amounted to 83 per cent. (1·5 ft.) of the expected figure.

The only Swedish contribution to this section was by Dr. B. Hellström, who described the compaction of a rockfill dam. A number of novel methods of procedure in consolidation were employed, all of which are discussed in the Paper. The settling resulting from these methods has been found to be almost negligible after the dam was completed, in February, 1950, until the present time.

Three Papers came from Yugoslavia. The first, by Messrs. Pavlin, Mladineo, Stubican and Nonveiller, discussed the Cetina river valley and the geological problems to which its development gave rise, with particular reference to the storage reservoir at Peruca. The Paper by Mr. Yevdjevic dealt with the construction of storage space in karstic regions (karstified limestone and dolomites) the geological features of which were set out at some length. The authors attempted to answer the question as to whether it was possible to justify the establishment of reservoirs in karstic ground, in spite of infiltrations of water around the dam.

The third paper, by Mr. Bogdan M. Rajcevic, dealt specifically with the Vlasina Dam, which is an earth dam 34·3 m. in height. The Yugoslav engineers' approach to the method of calculation for earth dams is set out in detail, and the figures given show that every possible success has attended the execution of designs according to the techniques described.

Professor E. Indri, from Italy, made a comparison

between stress data deduced from various methods of calculation, with particular reference to the Valle di Cadore Dam on the Boite Torrent, and showed that the elastic behaviour of the structure is well represented by results obtained according to the method employing elastic independent arches of various thicknesses.

From the other side of the world came a Paper by Mr. L. S. James, of New Zealand, who described the stresses and deflections found in the Maraitai dam, which is of the variable-radius arch type, on the Waikato river in the North Island of New Zealand. As in the Italian Paper mentioned above, the elastic arch method was used. The stresses obtained by this method and by a trial load analysis carried out by the United States Bureau of Reclamation are compared in the Paper.

The Relation of the Cement Content of the Concrete to Performance in Practice of (a) Gravity Dams (interior and exterior), (b) Arch Dams, (c) Buttress Dams), and its Influence on Permeability and Frost Resistance (Question 19)

In this section of the proceedings, the Papers from the different countries contained a considerable element of duplication, and it has been thought advisable to reproduce a rather fuller summary than would otherwise have been possible of the Report on Frost Resistance of Concrete and of the Definition of Concrete Mixes submitted to the Conference by the International Sub-Committee on Concrete for Large Dams, instead of attempting individual summaries of the Papers. This Sub-Committee consisted of 42 representatives from 21 countries, and within itself included 11 Papers on aspects of the subject described by the wording of Question 19.

The first Paper in the report dealt with methods of carrying out freezing and thawing tests on concrete. A questionnaire was circulated to 30 laboratories in many parts of the world and indicated that very wide differences in test procedures existed. The Paper then analyses the reports from these laboratories under the headings as follows:

A Details of Test Method*

(a) *General*

1 —Size of tests specimen and number tested.
2 —Method of saturation with water and control exercised on degree of saturation on specimens before freezing.
3 —Number of freezing and thawing cycles.
6a—Time per cycle.
6b—Special manipulation.

(b) *Freezing*

4a—Volume of freezing chamber; number and volume of specimens tested in the chamber at one time.
4b—Medium.
4c—Exposure of faces during freezing.
4d—Temperature to which cooled and where measured.
4e—Rate of freezing.
4f—Duration of freezing and time at minimum temperature.

(c) *Thawing*

5a—Medium.
5b—Temperature to which warmed.
5c—Rate of warming.
5d—Duration of warming and time of maximum temperature.

(d) *Test Methods*

7 —Property measured to assess damage and interval at which repeated. Method used for determining modulus of elasticity.
8 —Control specimens for comparison.
9 —Expression of results.
10 —Requirements for passing test.
11 —Variability.

B Details of Preparation of Specimens

12 —Concrete mixes.
13 —Cement used for tests on different aggregates.
14 —Aggregate used for tests on different cements.
15 —Admixtures.
16 —Method of making.
17 —Curing.
18 —Age when tested.

C General

19 —Other properties measured.
20 —Effect on frost resistance of:
(a) Size or shape of specimen.
(b) Rate of freezing.
(c) Duration of freezing.
(d) Water evaporated during freezing in air.
(e) Varying exposure of faces to frost.
21 —Preferred non-destructive tests.
22 —Effect of test conditions on relative order of frost resistance.
23— Correlation of test results with field experience.
24— Any other information or comment.

* The numbers refer to the respective sections of the Questionnaire.

This list is given in full in order to indicate the data which the Committee felt to be desirable to obtain if a full examination of this problem was to be attempted. Although it is impossible here to summarise the answers given to this long questionnaire, the list may be of service in indicating to those of our readers engaged in this aspect of hydro-electric engineering what types of data may be usefully accumulated in order that internationally agreed standards may be evolved.

The second Paper in the report dealt with the comparison of non-destructive methods of observation used in freezing and thawing tests of concrete, and it is stated that two methods of observation should be employed at the same time. These should include a method indicating superficial deterioration and a method indicating internal disintegration of the test specimen. This subject is also touched on in the third Paper in the report, while the fourth Paper deals with surface temperatures in concrete dams subject to frost. A large amount of information accumulated in Italy is included, extending over a period from 1930 onwards.

The superficial temperature of concrete is given in a Paper in the Report which comes from Electricité de France and some interesting diagrams show the distribution of temperature changes inside a concrete structure. There is also a British contribution to the Report on the same lines. The influence on the size of the aggregate on the strength of the frost resistance of concrete is discussed in another French contribution to the Report, again with a wealth of figures showing the results of cyclic freezing and thawing on test specimens. This subject is also touched on by Mr. Paul Klieger in a section contributed by the Research and Development Division of the Portland Cement Association, Chicago, U.S.A. Dr. Fritsch, Austria, described research into the laws governing the resistance to freezing and thawing of concrete mixtures carried out in a number of Austrian laboratories.

The last section of the Report of the Sub-Committee on concrete gives an extremely valuable definition which it recommends should be used in connection with the concrete mixes used in dam construction.

We quote here in full the list of minimum data required in a brief description of the concrete in a dam.

(Space prevents us from detailing the data which it is recommended should be required for a full report on the concrete, but this, of course, will be found in the paper.)

Minimum Data Required for a Brief Description of the Concrete in a Dam

(a) Cement content of finished concrete (in kg. per cu. m or lb. per cu. yard).

(b) Effective water/cement ratio (i.e. corrected for aggregate in the saturated, surface-dry condition).

(c) Type or types of cement with reference to any standard specifications applicable. (In the case of "blended" cements, the proportion by weight of pozzolana or other material present should be stated.)

(d) Broad petrological classification of aggregate, e.g., limestone, granite, etc., origin (i.e., crushed or natural), and maximum size employed.

(e) Method of compaction.

(f) Use of workability aid or air-entraining agent.

(g) Estimated air content of finished concrete, whether entrained or entrapped, as a percentage by volume.

Communications

Apart from Papers presented under the four main headings (Questions 16, 17, 18 and 19) there were a number of Papers, known as Communications, dealing with various aspects of civil engineering technique in the hydro-electric field.

Two Communications were sent to the Conference from Great Britain. The first, by Mr. H. G. Keefe, of Sir Alexander Gibb and Partners, described the investigation of stress distribution in a diamond head buttress dam by the use of photoelastic model tests. The tests were embarked upon first to provide a check on the design method of analysis as applied to an ideal material, and secondly to provide information, not obtainable by any other method of analysis, concerning the distribution of stresses within diamond buttress heads, including special heads perforated by slots, etc. Some interesting photograph illustrations are given and it was shown that the stress values obtained showed reasonable agreement with the stresses calculated for an ideal buttress of the same shape.

The second Paper, by Dr. G. A. Taylor, of Aberdeen University, and Mr. Oscar Elsden, of Sir Alexander Gibb and Partners, dealt with the detailed investigation of a right-angle bend at the foot of a vertical spillway shaft. This study related to the Strathvaich Dam under construction for the North of Scotland Hydro-electric Board, and the object of the study was to determine the discharge coefficient of the spillway, to study the flow conditions in the Kaplan bend, and to determine what modifications if any would be necessary or desirable.

Perhaps the most interesting of the four French Papers was that by Monsieur M. Peltier, of the Central Laboratory for Roads and Bridges, in which a description is given of a new type of apparatus for measuring the compressibility of soils. This consists of a triaxial cell holding the sample which is isolated in a rubber membrane. The cell is filled with water which allows a hydrostatic pressure to be applied, and the variations of the volume of the water sample can be measured by reading the movement of the water on a graduated tube extending from the apparatus.

The other three Papers related to the measurement of deformations in the Gage Dam, the drainage of the intake gallery of the Pralognan scheme and the Chaudanne Dam on the Verdon, where special consolidation and water-tightness procedures were necessary.

There were also three papers from Algeria; the first by Monsieur H. Duquennois, describing precision topographical methods produced in soil stability surveys, while the second Paper, by Monsieur Raymond Vormeringer, described in detail the construction of a flexible screen for the Iril Emda Dam on the Oued Agrioun, and the problems which arose during its connection to the rigid structures.

The third Paper, by Monsieur J. Raud, gave some interesting details about the measurement of permeability of a non-water-bearing soil by means of air injection tests.

A single Paper from Morocco by Messrs. J. Caille and P. Habib described a comparative study made on models of different types, of water leakages in the rock around the Bin-el-Ouidane Dam, using the electric analogy method. A three dimensional small scale model was made, and the conducting substance, which was in this case water, percolating through the sand model was studied according to the three dimensional electric analogy method. The results were extremely useful.

The subject of model tests is always of interest to hydro-electric engineers, and Ing. E. Fumagalli, of Italy, devoted his Paper to the study of appropriate materials used in preparing dam models for static and dynamic tests.

The two remaining Italian Papers under this heading, by Professor Ing. D. Tonini, dealt with the displacements at the top arch crown of the Pieve di Cadore Dam.

Spanish hydro-electric practice was represented by an interesting Paper by Professor Enrique Becerril which took the form of a commentary on modern Spanish practice. Professor Becerril showed that the hydro-electric future was much brighter in Spain now that a large programme of reservoir construction was almost complete, and it was hoped that there would be no longer any necessity to restrict power consumption. Drawings and diagrams show typical examples of many kinds of dams.

A somewhat similar Paper came from Portugal, prepared by Senhores Manuel Rocha, J. Laginha Serafim, A. Ferreira da Silveira, and O. V. Rodrigues. It dealt with the observations, made on nine dams, in regard to the measurement of the absolute displacements by the geodetic method, by the slide target method, and by precision levelling; and of relative displacements by means of plumb wires and of the movement of joints by means of strain gauges at the surface and joint meters in the interior of the concrete. Temperature measurements are also described, and the Paper illustrates the types of equipment used for these measurements.

The second Paper from Portugal, by two of the same authors, dealt with the use of model tests. It is concluded that many problems still remain to be solved. As with the Italian Paper mentioned above, they devote some part of their Paper to a discussion of the best types of material to use for model testing.

Three Papers from Yugoslavia dealt respectively with ideal soil conditions for the construction of earth dams; the isolation of a karstic gulf (a "ponor") in the Niksic Plain; and the problems of water storage in a periodically inundated karstic plain at Niksicko Polje.

The Austrian approach to stress research on dam

profiles (especially on gravity dams with a large longitudinal tunnel at the base) was given in a Paper by Dipl. Eng. Wilhelm Steinboeck. The optical stress testing method was adopted, a series of tests being carried out on models of two gravity dams of different heights.

A near neighbour of the author of the preceding Paper, Dr. Ing. Heinrich Press, of the German Federal Republic, also discussed stress tests, but on this occasion on an arch dam supporting a gravity dam. Model tests were carried out and are reported in detail in the Paper.

Four Papers came from Switzerland. The first, in which no author's name is mentioned, dealt with the famous Grande Dixence Dam, which is 932 ft. in height, constructed by raising the level in successive stages. In the first stage, having a height of 594 ft., a stepped downstream face has been employed, while the second stage consists of adding vertical columns or blocks on to these steps; each successive stage will be constructed in the same way, and it is expected that the dam will be completed over a period as long as 16 years, ending in 1967. This dam is of a totally exceptional nature, and the Paper deserves close study.

A Paper by Professor E. Paréjas and Ing. O. Rambert dealt with the Mauvoisin project at Valais, and described some interesting and novel features of the grouting methods used for dealing with leakages discovered by very accurate preliminary geological reconnaissance.

The determination of the modulus of elasticity of rock by means of loading tests in circular galleries was the subject of a Paper by Messrs. A. Preissmann and B. Gilg. It was shown that the methods developed by the authors gave results which lined up with those found in practice, and which were seen to be generally satisfactory.

The final Swiss Paper, by Messrs. Henri Gicot and Pierre Lardy, again touched on model testing and dealt with an asymmetrical arch dam related mainly to the project at Lienne, in Switzerland, which is 525 ft. high. In this case the authors stated that the experimental method and the analytical method must work side by side, and that both must be employed.

The single Swedish Paper, by Mr. Valter Faruskog, of Stockholm, dealt mainly with the cutoff wall connecting the front part of a concrete gravity dam founded upon rock, to the rock itself. The suggestion was made that the tightening element should be separated by a permanent joint from the rest of the dam, whether it takes the form of a cutoff wall or an apron covering the rock in front of the dam.

Of the two Communications from Turkey, the first, by Dr. Ing. Necati Engez, dealt with a subject not so far touched upon in the Communications section. He dealt with the sedimentation studies carried out in the Elmali dam reservoir, now 60 years old. The second Paper, also by Dr. Engez, described the Turkish approach to a second dam at the same site, this time taking the form of a round head buttress type concrete structure, with a height above foundations or 47·5 m.

Turning away from Europe, there were two Communications from the United States and one from Mexico. Mr. W. A. Dexheimer, Commissioner of the Bureau of Reclamation, described a series of nine proposed dams on the upper Colorado River. These are all very large dams and have widely differing characteristics; they are described in the Paper and are illustrated by artists' impressions.

Mr. Roy R. Clark brought new light to bear on concrete control measures aimed at producing a uniformly stable condition at the base of a dam. His study was mainly related to the Detroit Dam, and described the very great care with which inspection was carried out during construction.

The Mexican Paper gave a lengthy survey of various earth-fill dams built in Mexico, and special reference was made to the types of spillway designed for these structures.

An interesting Paper from India, by Mr. D. Doddiah, head of the Mysore Engineering Research Station, described a type of construction which is widely used in India—the siphon; and the author set out the basic design principles for a number of types of siphon in use there.

Mr. Khaja Azeemuddin, the Chief Engineer of the Karnafuli and Narsak Dam Projects, Pakistan, described the 160 MW project at Karnafuli, which involves the construction of a dam 130 ft. high with a storage capacity of 5·4 million acre ft. Many difficulties were encountered in connection with the foundations as the dam is sited in a region of geologically young sedimentary rocks consisting of various grades of shales and soft sandstone. Borings totalling nearly 10 miles in length were undertaken to ascertain the sub-surface conditions.

There were two Papers from Japan. Mr. Shunsuke Takano, of the Onoda Central Research Laboratory, dealt with the effect of temperature on the strength of mass concrete. He observed that the rate of strength development of cement paste made at low temperature was strikingly greater than the same paste made at high temperature, and reached the conclusion that the placing of concrete at the lowest possible temperature above freezing point produced favourable effects on the development of the strength. Also, the placing of mass concrete at low temperature not only reduced the maximum temperature in the interior, but also developed greater strength at a later age, thus creating more favourable conditions for avoiding temperature cracking.

The second Paper took the form of a Communication from the Arch Dam Committee of the Tokyo Electric Power Company, and gave a lengthy survey of a series of exhausive tests carried out on the Naramata Arch Dam, having a height of 12·8 m., constructed as a coffer dam for the main Sudagai Dam of the Power Company. The Committee conducted foundation rock tests, temperature measurements, deflection tests, vibration tests, stress measurements and model tests, and these are all set out in full in the Paper.

(To be continued)

Mr. Philip Emanuel. We deeply regret to record the death, on July 15, in his 74th year of Mr. Philip Emanuel, a director of Tothill Press Limited, the company owning and publishing this journal. Mr. Emanuel had been a director of Tothill Press Limited since August, 1945, and had also been a member of the Board of the parent company, Odhams Press Limited, with which he had been actively associated for over 40 years. Cremation took place at Golders Green on July 18 and a Memorial Service was held on July 21 at the Liberal Jewish Synagogue, St. John's Wood Road, N.W.8.

Correspondence

Data for Preliminary Estimates

To the Editor of WATER POWER

Upon reading Mr. Nag's and Mr. Madhavan's interesting article on this subject in your June issue, the dimensions of the concrete scroll case according to Fig. 7 seemed to me to be smaller than in present-day practice. In the majority of modern power stations working at heads up to 90 ft., propeller or Kaplan type turbines are installed. A general dimension sketch for a concrete scroll case should therefore be worked out for these high-specific-speed turbines.

Having tabulated and examined a number of European and American Kaplan turbine installations

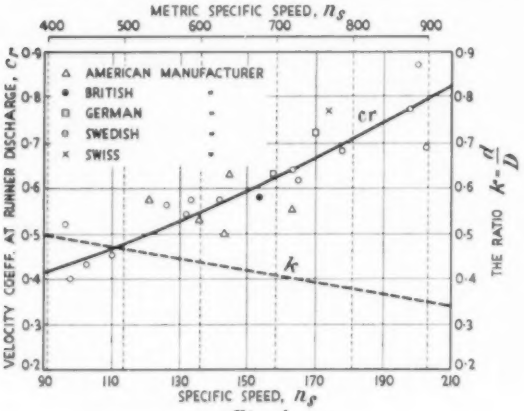


Fig. 1

for working heads up to about 100 ft., it is evident that a unit area of $2 \times 1{\cdot}4 = 2{\cdot}8$ taken from Fig. 7 holds true only at comparatively low specific speeds.

Assuming a width of K_1D and a height of K_2D of the spiral inlet section the following equation will be obtained for determining the unit area K_1K_2

$$c_i\sqrt{2gh}K_1K_2D^2 = c_r\sqrt{2gh}\frac{\pi D^2}{4}(1-k^2) \quad \ldots\ldots\ldots\ldots \quad (1)$$

where

c_i = velocity coefficient at the inlet of the scroll case
c_r = axial velocity coefficient at the runner discharge
D = runner discharge diameter
d = diameter of runner hub at the discharge
k = d/D ratio of hub and runner diameter at the discharge.

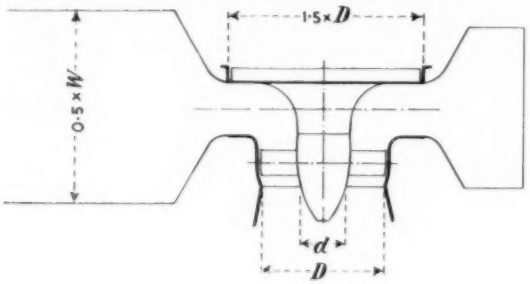
Regarding the inlet velocity $c_i\sqrt{2gh}$ of the scroll case the manufacturers have selected coefficients ranging from 0·08 to 0·11 for the plants examined. All the coefficients have hereby been calculated on the assumption of a turbine efficiency of 85 per cent. With an average value $c_i = 0{\cdot}092$ the equation for the unit area can be represented in the following form,

$$K_1K_2 = \frac{c_r}{0{\cdot}092}\cdot\frac{\pi}{4}(1-k^2) = 8{\cdot}55\,c_r(1-k^2) \quad \ldots\ldots\ldots\ldots \quad (2)$$

The axial velocity coefficient c_r increases and the coefficient k decreases with the specific speed n_s. In Fig. 1 these coefficients are plotted against n_s. The average ratio K_1/K_2 of the installations studied is approximately equal to $K_1/K_2 = 2$ and by inserting this value in equation (2) the unit scroll case width will be obtained from,

$$K_1 = \sqrt{2 \times 8{\cdot}55\,c_r(1-k^2)} = 4{\cdot}14\sqrt{c_r(1-k^2)} \quad \ldots\ldots \quad (3)$$

For a certain given specific speed the unit width K_1 can be taken from Fig. 2 assuming as mentioned before that the height of the spiral is half of the width $K_2 = K_1/2$. The section and plan view of a typical concrete scroll case is shown in Fig. 3. With the use of Figs. 2 and 3 the spiral dimensions may be estimated. The dimensions thus obtained are only approximate, but they are sufficiently close to warrant the use of the sketch for preliminary estimating purposes. A comparison of estimated and actual dimensions for existing plants shows that the average error of the linear measurements will amount to ± 6 per cent.



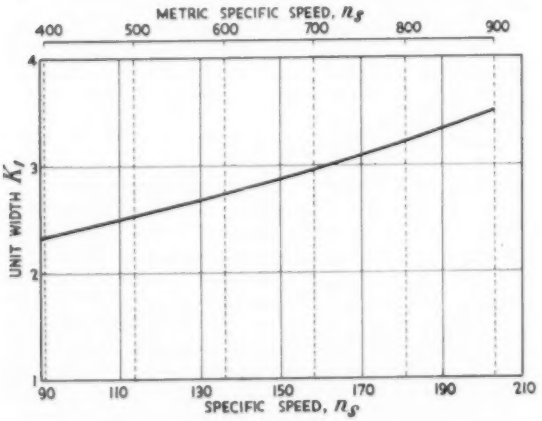


Fig. 2

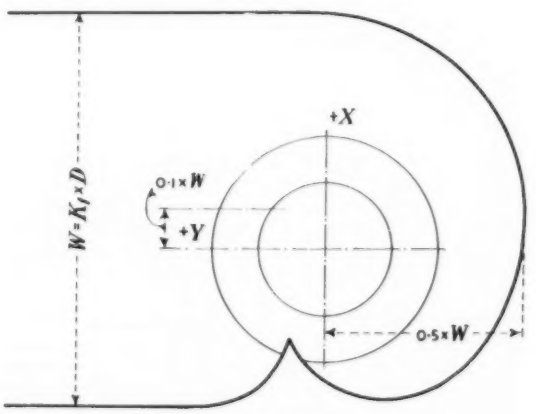


Fig. 3

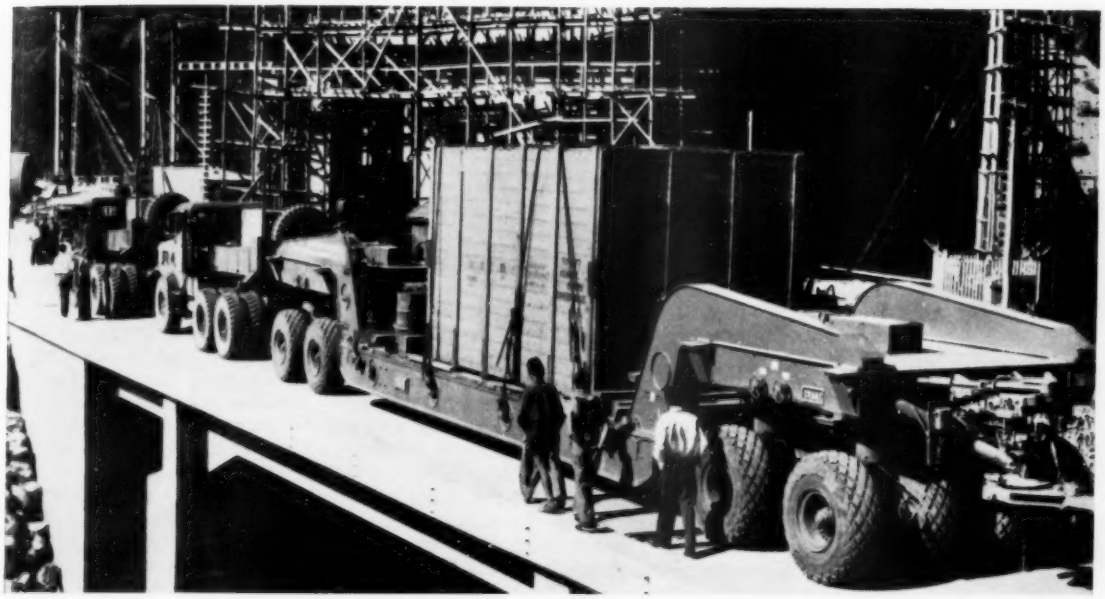
On the other hand using the sketch according to Fig. 7 in the above-mentioned article the average error will be – 28 per cent.

According to American practice the spiral tip is often placed on the downstream side of the X – line. The eccentric location of the turbine can then be made smaller as less water will be passing through the section at + X.

GUNNAR ENGLESSON

Trollhättan, Sweden

Transport of Heavy Equipment



120 ton trailer built by Cranes (Dereham) Limited, Norfolk, carrying an 80 ton rotor to Munyang powerhouse in the Snowy Mountains

A notable piece of road transportation was carried out in Australia recently by the Snowy Mountains Hydro-Electric Authority when an 80 ton rotor was hauled to Munyang powerhouse. Before this achievement was practicable some 80 miles of new access road had to be constructed to connect the various work sites and a further 100 miles of existing roadway had to be reconditioned. Large tracts of the Snowy Mountains lie at an elevation of about 7,000 ft., the terrain having originally been a plateau which was subsequently eroded until it is now traversed by ravines and deep valleys, so that the route is by no means an easy one. The road train consisted of a 120-ton capacity trailer unit built by Cranes (Dereham) Ltd., Norfolk, and two Thornycroft "Mighty Antar" trucks. The trailer, which was fully described in our July 1953 issue, page 276, is 67 ft. long and has a weight of 40 tons, including the hydraulic system incorporated for both steering and suspension purposes. The suspension allows each leg of either bogie to be lifted independently and so facilitates the replacement of a damaged tyre, compensates for any unevenness in the road surface and enables the load to be tilted when negotiating an unfavourable curve or a low-arched bridge. It also, of course, affords the means of raising the load clear of the ground and vice versa The two tractors, each of which is fitted with a 250 h.p. motor, may be linked as a pushing or pulling team and can consequently be used fore or aft of the trailer. For these data we are indebted to *Road International* and to the Goodyear Company, whose tyres are fitted to the transporter.

IEC Specification for Porcelain Insulators. Publication 75, issued by the International Electric Technical Commission, applies to insulators of porcelain or similar ceramic materials for overhead electric lines with a nominal voltage of 1,000 and upwards. It is divided into six chapters:— Chapter I, General; Chapter II, Definitions used in the specification; Chapter III, General rules for impulse tests, power-frequency tests and classification of tests.

The classification of tests makes a distinction between three groups of tests; Group I, which is defined as those tests which are intended to verify those characteristics of the insulators which depend only on the shape and size of the insulator and of its accessories; Group II, which contains tests for the purpose of verifying the other characteristics of the insulator and the quality of the materials used; and Group III, which contains tests for the purpose of eliminating insulators with manufacturing defects.

These tests are fully described and there are two appendices, the first dealing with humidity correction factors for flashover voltages while the second deals with the method of producing artificial rain.

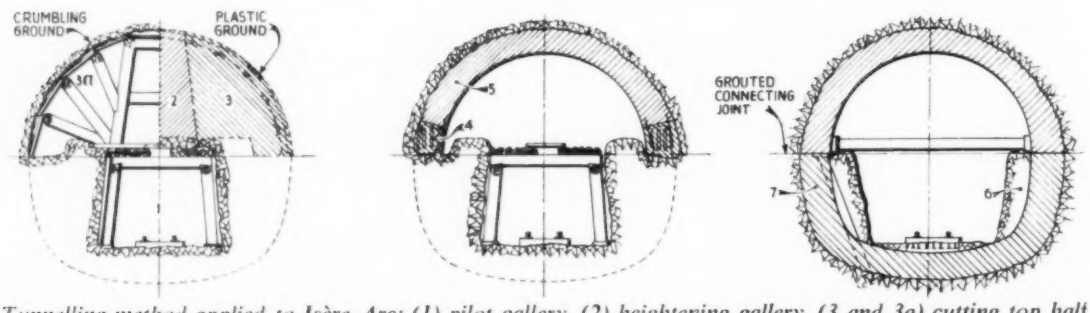
Abstracts from the World Technical Press

The Isère-Arc Tunnel

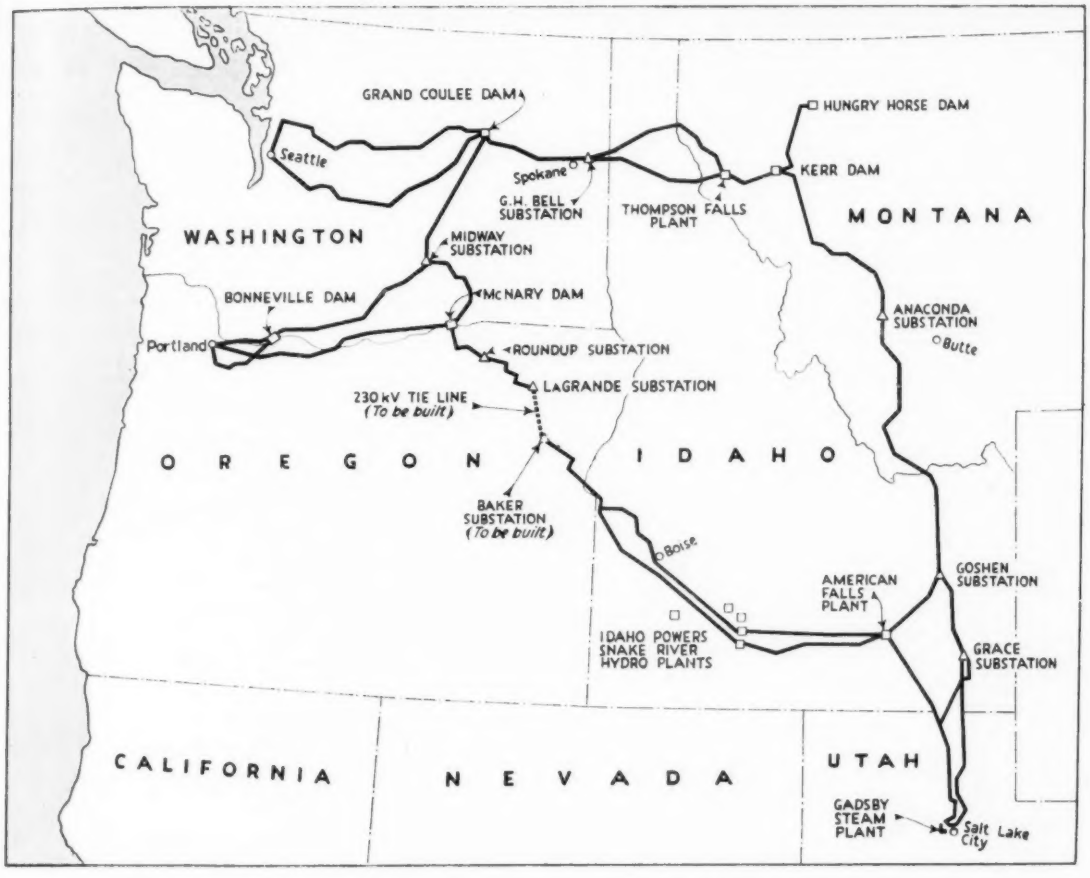
A comprehensive account is given of the methods applied to the construction of the main power tunnel driven across the Grand Arc massif to feed the turbines of the Randens power station. With its total length of 11,694 m., including two construction adits 1,000 m. long upstream and 100 m. downstream respectively, it is, in this order of magnitude, the first tunnel built since the construction of the Loetschberg Tunnel in 1913, and its lined cross-section of 32·4 sq. m. net, is but slightly smaller than that of the major transalpine double-track railway tunnels. The superincumbent rock has a maximum thickness of 1,800 m. about halfway between the two adits, a height second only to the 2,200 m. of the Simplon; it is obvious that at such a depth below ground working conditions are exceptionally difficult. Both adits were fully equipped to meet all requirements, viz., air-compressing plant, ventilating fans, repair shops, a 1,500 kVA transformer station, drainage pumps, a heated changing-room for the workers, etc. The drilling, lining operations are described at length. Tunnelling proper began in March, 1949, at the upstream adit, the downstream adit being started a little later. The latter was, in consideration of its small length, driven to the full cross-section of the power tunnel proper, while the section of the upstream adit had the same width but a smaller height. Supporting steel arches and roof anchoring by means of Ancral expansion bolts were extensively used. In spite of all the difficulties encountered, the last block of rock separating the two sections was blasted on March 23, 1953, the divergence at the connecting point being no more than 28 cm. in direction and 3 cm. in floor level. In their conclusions, the authors advocate the application of roof bolting in two different circumstances—in faulted strata, in which the bolts ensure a closer bonding of neighbouring blocks; and in highly schistose strata, in which the bolts bind the rock flakes together Steel arches incorporated in the concrete lining are, however, to be preferred when the tunnel crosses faults. The advantages of full-section tunnelling, especially in strata expanding under decompression, is also underlined, and particular stress is laid on the importance of an efficient and reliable ventilation in long tunnels of this kind. (M. Olivier-Martin and M. Kobilinsky, *La Technique Modern-Construction*, Vol. X, No. 4, April, 1955, p. 145, 12 pp., 14 ff.)

Bou-Hanifia Dam

The results of measurements and observations made at the Bou-Hanifia rockfill dam in Algeria between 1940 and 1951 are reviewed and discussed. After a description of the measures taken to ensure the imperviousness of the structure, their efficiency is demonstrated by graphs showing the amount of seepage collected at the toe of the dam in relation to the fluctuations in the water level of the reservoir. In consideration of the large infiltration area offered by the structure (exclusive of toe wall, 41,000 sq. m., of which about a third in permeable sand), these losses are very small and, even during the initial period of 1941, never reached the estimated maximum. The analysis of the losses is followed by an investigation of the deformation of the ground and of the rockfill, and the behaviour of the bituminous-concrete facing. The stresses in the toe wall were controlled by means of 51 expanding and sound-giving telltales of the Coyne type. Generally speaking, the deformation and compaction (settling) curves show that the greatest movements occurred while the rockfill was being placed; in actual operation, these deformations are substantially reduced but are nevertheless considerable enough to justify all the work done to ensure a maximum flexibility of the upstream facing. On the whole, the observations made since the impounding of the reservoir in 1940 established the excellent behaviour of all the parts of the dam which have combined to ensure imperviousness, i.e., grouting screen, toe wall, and upstream facing. In his conclusions, the author suggests the following improvements in the construction of rockfill dams: (1) Deep toe-walls, such as exist at Bou-Hanifia, could be advantageously replaced by multiple grouting screens, which are cheaper than concrete walls 2 to 3 m. thick, provided that control piezometers are fitted upstream and downstream of the screen; this applies particularly to dams provided with a watertight core; a moderately high toe-wall still remains essential, however, in permeable ground. (2) The flexibility of the upstream facing at low temperatures should be increased; hand-packed selected rockfill, such as applied at Bou-Hanifia, reduces the deformability of the structure but costs twice as much as dumped fill. A more flexible upstream facing would permit the adoption of a less expensive method of fill placing. (3) Heights being equal, the volume of a rockfill dam with a stiff



Tunnelling method applied to Isère Arc: (1) pilot gallery, (2) heightening gallery, (3 and 3a) cutting top half sections, (4 and 5) cutting roof, (6 and 7) cutting and concreting invert and jambs in alternate sections



Final link between east and west groups of the North-west Power Pool

facing (aggregate batter 2·35) is three times as much as that of a gravity structure (batter 0·8), but the cost of the placed fill varies between one-half (fill gained by blasting and hand-packed) and one-fifth (fill obtained from debris deposits and dumped) of the price of concrete. In the case of rockfill extracted from quarries, it ought to be possible to reduce costs by deep drilling and large blasts. (J. C. Ott, *Bulletin Technique de la Suisse Romande*, Vol. 81, No. 4, 19.2.1955, p. 57, 11 pp., 13 ff.)

New Link in U.S. North-west Power Pool

The final link between the east and west groups of the North-west Power Pool will be completed by May, 1956, in virtue of a recently signed agreement between Bonneville Power Administration and Idaho Power Co. According to this agreement, Idaho Power Co. is to build in Oregon a 50-mile, 230 kV transmission line from Baker to La Grande substation of the BPA for interchange of energy. The link will provide a second and more direct route for power flow between the Utah-Idaho and Washington-Oregon groups of the Pool; once in operation it will ensure a more efficient use of NW power as BPA will be in a position to supply surplus energy in summer from the Federal generating system, when the Snake river flow is low and irrigation pumping loads are heavy; in reverse, Idaho Power will deliver energy in winter, when Columbia river flow is at its lowest (*Electrical World*, Vol. 143, No. 15, April 11, 1955, pp. 18/19, 1 map.)

Baygorria Project, Uruguay

Proceeding from the conditions created by the setting up of the first stage in the Rio Negro Development at Rincon del Bonete (*WATER POWER*, Nov., 1951, p. 411), the various aspects of the Baygorria Project, downstream of Rincon del Bonete, are discussed. After the suitability of the dam site selected had been confirmed, a storage level of +54·00 m. was finally accepted for the Baygorria reservoir; this is not quite the theoretical optimum but a higher level was out of the question in consideration of the backwater level at the little town of Paso los Toros. Extensive boring established that geological conditions were considerably better than at Rincon del Bonete, and investigations based on measurements taken in the course of many years showed that hydrological prerequisites were the same as at Bonete. The river flow at Baygorria was assessed at 10 per cent. higher than at Bonete, after evaporation from the 100 sq. km. Baygorria reservoir had been deducted. As the storage arrangements at the Bonete plant are not yet complete the discharge from the turbines could not be

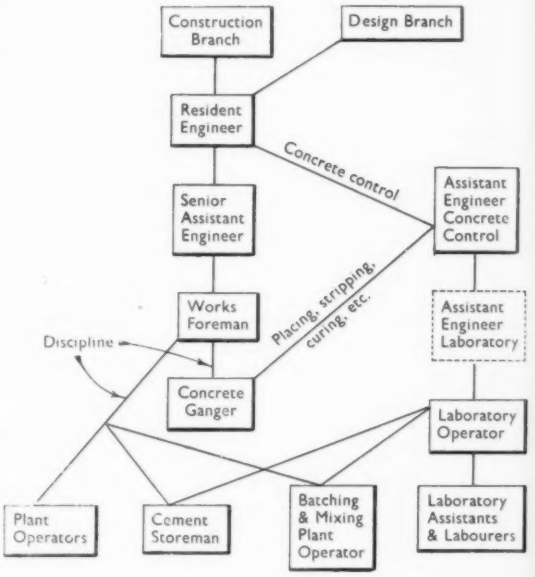
used as a basis for Baygorria, and it was found more suitable to take the regulated flow calculated from the total natural discharge. Bonete and Baygorria are in fact to share proportionally in the total load occurring at any given time, and—in relation to the Montevideo thermal plants—are to be considered as a single production unit. The Baygorria power house will have three generating groups, instead of four as at Bonete, and each of its turbines, when open to the full (exceeding the cavitation limit at a head under normal), will be able to pass 273 instead of 240 cu. m. per second, so that the maximum workable discharge will reach 819 cu. m. per second. Pending the setting up of the contemplated downstream Puerto plant, it is estimated that the Baygorria plant will be able to attain a yearly mean production of 570 million kWh. Deducting from this figure the loss of 50 million kWh caused by backwater at Bonete, the net production will be 520 million kWh or, measured on the outgoing side of the Montevideo transformer station, 440 million kWh. The power house is projected on the left bank of the river, and prolonged in a straight line by the spillway block and the massive dam extending to the right bank. Adjacent to the power house will be a large outdoor switchyard with the three transformers. To avoid the inconvenience observed at Bonete, where fine sand blown from outside is apt to penetrate the machines, the machine hall will be hermetically closed and artificially ventilated. Since, as in Bonete, there is practically no danger from floating ice or large drifting debris, the nine 14 × 11 m. spillway bays will be controlled by cheap, simple Taintor gates of which the one nearest to the power house will be fitted at the top, as a precautionary measure, with a hinged shutter. An alternative provides for a five-bay spillway controlled by five double-hook type gates 21 m. in width by 14 m. in height. In both alternatives, the spillway, with all its bay fully opened, can pass catastrophic floods of 9,000 cu. m. per second without affecting the storage level of + 54 m. The dimensions and layout of the whole plant were worked out on the basis of extensive model tests carried out at the Theodor Rehbock River Works Laboratory of the Karlsruhe Technical University. (Adolph Ludin, Berlin, *Die Wasserwirtschaft*, Vol. 45, No. 3, December, 1954, p. 57, 8 pp., 17 ff.)

Air-operated Crane Hook

An air-operated crane hook has cut labour costs and increased safety in the placing of concrete at the Army Engineers' Oahe dam across the Missouri river near Pierre, S.D. The hook, designed by William Mittry of Mittry Constructors, and fabricated in the firm's shop on site, has a capacity of 10 tons and roughly resembles an old-fashioned pair of ice tongs: it has two semi-circular arms that overlap each other and interlock with two air-actuated pistons to open and close them. The air hose leads to the crane cab where the operator works the controls. As a safety measure against dropping a bucket, the hook can be disengaged only when not carrying a load. The new hook eliminates a man whose sole duty was to stand atop a truck to engage and disengage the conventional hook by which the four-yard bucket was lifted. There is no possibility of either a man's hand being caught between the bail of the bucket and the hook, or a swinging bucket knocking the man off the truck. (*Engineering News-Record*, March 10, 1955, p. 48.)

Concrete Control at Burrinjuck Dam, N.S.W.

After its completion in 1928, the Burrinjuck Dam, an irrigation water conservation structure on the Murrumbidgee River, began to exhibit several undesirable features. Remedial measures were carried out between 1949 and 1951 and included a heightening of the dam wall, increasing the cross section, and a reconstruction of the spillway with a view to ensuring a major pre-flood peak discharge. This required the placing of 96,000 cu. yards of concrete, both mass and reinforced. The original paper deals in detail with the procedure and supervision of the manufacture and



placing of the concrete and the evaluation of the variables affecting the concrete strength, based on the results of three years' testing. After a description of the works, of the testing laboratory, and of the various tests themselves, the author makes various recommendations on control methods for low-heat cement and, because of the predominance of cement variation on concrete strength, advocates the tightening of cement specifications hitherto regarded as acceptable. The standard cement mortar test has many unsatisfactory features, and a more plastic cement may give more consistent results. It is the scatter of cement strengths which plagues the concrete engineer more than low strength, and in small jobs, where cement testing cannot be carried out, concretes can drop greatly in strength with no apparent explanation, simply because job mixes were unknowingly derived when the cement supplied happened to be of high strength. The fundamental improvement, therefore, calls for cement of more uniform quality. The chart appended indicates how concrete control at Burrinjuck dam has been made independent of construction pressures, while close personal supervision of all details of batching, mixing, placing and curing, is still retained. (J. M. Muir, B.E., *The Journal of the Institution of Engineers, Australia*, Vol. 26, No. 7-8, July-August, 1954, p. 151, 8 pp., 7 ff.)